

Guidelines for the Use of Structural Precast Concrete in Buildings



NEW ZEALAND CONCRETE SOCIETY



NEW ZEALAND SOCIETY FOR
EARTHQUAKE ENGINEERING



Centre for Advanced Engineering
University of Canterbury, Christchurch, New Zealand

Guidelines for the Use of Structural Precast Concrete in Buildings

Second Edition

Report of a Study Group of the

New Zealand Concrete Society

and the

New Zealand Society for Earthquake Engineering



Centre for Advanced Engineering
University of Canterbury Christchurch New Zealand

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— Second Edition —

ISBN 0-908993-20-X

Printing History

First published August 1991

Reprinted June 1992

Second Edition December 1999

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Publisher

Centre for Advanced Engineering
University of Canterbury
Private Bag 4800
Christchurch
New Zealand

Printing

Wickliffe Press, Christchurch

Cover Design

Hudson Design, Christchurch

Editorial Services, Graphics and Book Design

Charles Hendtlass and Úna O'Grady, Centre for Advanced Engineering

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Acknowledgements

Contributors — Second Edition (1999)

D K Bull (Editor), Holmes Consulting Group Ltd, Christchurch

D P Barnard, retired (from Cement and Concrete Association of New Zealand)

B J Brown, Fraser Thomas Ltd, Auckland

R C Fenwick, University of Auckland

D C Hopkins, Sinclair Knight Merz Ltd, Wellington

L G McSaveney, Firth Industries Ltd, Auckland

R Park, University of Canterbury

J Restrepo, University of Canterbury

R G Wilkinson, Holmes Consulting Group Ltd, Christchurch

Study Group — First Edition (1991)

A W Charleson (Editor), Victoria University of Wellington

D C Hopkins (Chairman), Sinclair Knight Merz Ltd, Wellington

D P Barnard, Cement and Concrete Association of New Zealand

B J Brown, Fraser Thomas Ltd, Auckland

D K Bull, Cement and Concrete Association of New Zealand

R C Fenwick, University of Auckland

J L Lumsden, Centre for Advanced Engineering

L G McSaveney, Firth Industries Ltd, Auckland

B Newsome, Downer and Company Ltd

R Park, University of Canterbury

R G Wilkinson, Holmes Consulting Group Ltd, Christchurch

Financial Support and Editorial Services

Acknowledgement is made of financial support for the two editions of this publication from the Centre for Advanced Engineering at the University of Canterbury, the New Zealand Concrete Society, the New Zealand Society for Earthquake Engineering, and the Cement and Concrete Association of New Zealand. Final compilation, editorial services, graphics and book design have been carried out by the Centre for Advanced Engineering.

Photographic Material

Many photographs from the first edition have been reproduced in this edition and the publishers are grateful to the following organisations for making photographic material available:

Firth Stresscrete Ltd, Holmes Consulting Group Ltd, Kerslake and Partners, Kingston Morrison Ltd, Murray-North Ltd, University of Canterbury, Victoria University of Wellington, Wilkins and Davies Ltd and Works Central Laboratories.

Chapter 1

Introduction

1.1 General

This is a second edition of this book, which was first published in 1991. The second edition came about through the need to incorporate relevant research undertaken during the first half of the 1990s, to bring some of the technical aspects in to line with the 1995 revision of the Standard for the Design of Concrete Structures (NZS3101:1995), and to respond to the continuing demand for the Guidelines that outstripped the quantity published, including the second printing.

Since the introduction of seismic design requirements in 1935, New Zealand has favoured reinforced concrete as a building material. Developments in its use have been related to successive changes in seismic design requirements, notably in 1965 and 1976. These changes were summarised by Park [1.1]. Further significant changes occurred with the publication of a code of practice for general structural design and design loadings for buildings [1.2].

In New Zealand, general design provisions for reinforced and prestressed concrete structures are contained in the code for general structural design and design loadings for buildings, and in a code for the design of concrete structures [1.3]. For buildings, the codes contain comprehensive provisions for the seismic design of cast-in-place concrete structures, but do not have provisions covering all aspects of precast concrete structures. Nevertheless, significant developments in the use of precast concrete have been made in spite of the fact that some aspects of the seismic design of precast concrete building structures have not yet been formally codified. This reflects an on-going innovation by New Zealand practitioners that will no doubt continue.

Since the early 1960s there has been a steady increase in the use of precast concrete for structural components. Precast concrete fabricators have developed skills to meet the increasing demand, using their experience with increasingly popular “non-structural” cladding units. The use of precast concrete in flooring systems very rapidly became commonplace, leaving cast-in-place floor construction generally less common and uncompetitive. However, until the late 1970s

to early 1980s, the use of precast elements for seismic resistance in moment resisting frames and walls was the exception rather than the rule.

The boom years of the mid-1980s produced a significant increase in structural applications of precast concrete, which had the advantages of familiar materials and methods, high-quality factory made units, and speed of construction. Time and resources available for experimental verification were significantly reduced.

With high interest rates and pressure for new space, the advantage of speed gave precast concrete a distinct cost advantage. Designers quickly responded, as did fabricators. Above all, the number of designers specifying precast grew rapidly, and the ranks of fabricators swelled to meet demand.

Contractors quickly adapted to demands of economical precast assembly with enhanced craneage and construction techniques, and maximized off-site fabrication to compensate for a shortage of skilled labour. There was pressure to perform.

The result was a remarkable development in all aspects of the structural use of precast concrete as all sectors applied their ingenuity to obtain a competitive edge. Whilst design aspects were generally carefully considered, the solutions proposed normally assumed that extrapolations from testing of cast-in-place specimens would be valid. In some instances, especially in floor construction, the monolithic integrity of cast-in-place concrete was not fully replaced.

With this increase in the use of precast concrete structural elements came an increasing concern that some of the design solutions being used should be more fully researched. Even if there was no reason to doubt the validity of extrapolating cast-in-place results, the number of major buildings employing precast concrete for seismic resistance demanded that more research and testing be done to justify confidence in the structural systems.

1.2 Formation of Study Group

In February 1988, a seminar at the University of Canterbury, attended by designers, researchers, fabri-

cators and constructors, highlighted a growing need to investigate and verify the performance of precast concrete in structural members for seismic resistance.

Following the seminar a study group jointly-funded by the New Zealand Concrete Society, the New Zealand Society for Earthquake Engineering, and the Centre for Advanced Engineering at the University of Canterbury, was formed. Members were selected to represent the design, research, fabrication and construction aspects. Its objectives, as with previous study groups, have been:

- to bring together and summarise existing data;
- to present the data in a form useful for New Zealand conditions;
- to identify any special concerns;
- to indicate recommended practices (and draw attention to practices that are not recommended); and
- to recommend topics requiring further research.

Principal areas to be covered were:

- precast beams (both shell and solid), precast columns and their jointing;
- beam-column joints, especially if cast-in-place between precast elements;
- support and continuity of floor slabs;
- jointing techniques and connectors, constructability and tolerances;
- diaphragm actions; and
- behaviour of precast concrete wall systems (subsequently included).

Within each area, investigations focused on aspects that could possibly lead to different behaviour when compared to cast-in-place construction. Special consideration was given to related matters, including fire resistance, beam elongation, robustness, integrity and workmanship.

For this second edition, those responsible for contributing to the first edition were asked to make necessary changes reflecting developments since 1991. The original Study Group was not reconvened, but John Lumsden of CAE co-ordinated the individual responses.

1.3 Scope of the Guidelines

The scope of these guidelines follows the areas identified for investigation listed above. Generally, the

emphasis is on building structures rather than civil engineering structures. Furthermore, only structural elements are dealt with since architectural (non-structural) precast concrete is not normally designed to contribute to the overall structural integrity and requires a different set of design criteria. Although the focus is on seismic aspects, many sections refer to gravity load effects as well as volume changes such as creep, shrinkage and thermal actions, since these effects can result in a significant reduction in seismic performance. Durability of precast concrete is not included in the scope. Readers are referred to reference 1.3 for further information.

Each chapter contains references to overseas experience, research and testing. Overseas research results have required careful interpretation to allow for New Zealand's specific demands for ductility and capacity design, and for this reason much potentially valuable work could not be included with confidence. Interest in this subject is evident in work carried out in the USA, Japan, China, Romania and other countries. In addition, the Prefabrication Commission of the FIP has sets of recommendations which contain much relevant material [1.4, 1.5]. Where appropriate, overseas material is referred to and referenced in the various sections.

1.4 Summary

Floor Unit Support and Continuity

This chapter highlights the need for careful detailing to provide adequate continuity and maintenance of load paths through the structures. Typical floor unit support details are examined and their advantages and disadvantages described. Many details are seen to be sensitive to fabrication and construction tolerances. Details that reduce this sensitivity are indicated, notably the inclusion of continuity steel across beams.

The potential for beam elongation due to yielding of steel in plastic hinges is seen as a particularly important consideration for precast systems. Such lengthening could result in loss of support for precast floors. Guidance is given on this topic, utilising the considerable research that was undertaken in the early 1990s.

Frame Connections

Common assembly systems for moment-resisting frames are examined in detail and their advantages and disadvantages noted. Extensive reference is made to laboratory tests and illustrations of recent projects. Most are "monolithic systems" that seek to reproduce the essential features of cast-in-place systems. General indications are that with proper care in detailing, and in

construction, adequate performance is achieved by all methods.

Structural Wall Elements

These are examined under two headings: “monolithic” and “jointed” wall systems. “Monolithic” systems seek to reproduce a cast-in-place condition with strong joints between elements. “Jointed” wall systems, on the other hand, behave as a discrete number of precast elements with ductile connections.

Examples of details and applications of monolithic wall systems are given and a cautionary note included regarding the use of jointed wall systems.

Diaphragms

Cast-in-place diaphragms generally provide a comforting degree of integrity and continuity. Precast concrete diaphragms rely on connections between individual elements, with typically only reinforced concrete topping providing continuity. As a result, diaphragms incorporating precast elements need special attention to the functions they are required to perform. These are analysed and described and include transmission of shear, and resistance to volume changes.

Guidance is given on detailing for both the Serviceability and Ultimate Limit states.

Some potential dangers are described, especially in relation to hollow-core floors where there is a reduced shear area available at mid-depth. Beam elongation is highlighted as a further factor that can reduce the effectiveness of connections between floor diaphragms and supporting beams.

Grouted and Welded Bars

Techniques of anchoring bars are vital to the success of joints between precast members. Detailed guidance is given in all common methods, with particular emphasis on practical aspects. Advice is given on grout selection and on the large number of factors that influence it. Techniques for successful grouting in various common situations are described, including vertical, horizontal and sloping bars. A special section is devoted to grouting in specific situations such as beam-to-column joints and beam-to-beam joints.

Emphasis throughout is on achieving necessary quality, with a separate section on quality assurance.

A comprehensive chapter giving guidance on the welding of reinforcing bars is included. Welding, when properly controlled, is a practical method for joints in precast members. Special care is needed to account for tolerances, varying materials, site conditions and corrosion protection.

Embedded Steel Connectors

Common types are described and their importance of detailing stressed. Many details are seen to be particularly sensitive to variation in component dimensions and the proximity of other connectors. Attention to fire resistance and corrosion protection of such connections is emphasised.

Tolerances

Close control of dimensions has allowed successful jointing of precast concrete members. Production, erection and interface tolerances are described and recommended values given. It is important to recognise that the tolerance values needed to achieve many of the precast systems need to be much tighter than those quoted. It is vital that designers make it clear what tolerances are required to meet design assumptions so that fabricators and erectors are fully aware of the implications. Close co-operation between designers, fabricators and erectors at an early stage is essential.

1.5 Conclusions

Research carried out since 1991 has generally confirmed the integrity of the procedure given in the first edition. However, concern for the effects of tolerances and possible beam elongation has heightened the need for attention to these aspects.

The Study Group remains confident that precast concrete can be used successfully in earthquake resistant structures. However, it is essential that careful attention is paid to:

- conceptual design;
- detailed design;
- fabrication;
- transport;
- erection;
- jointing;
- durability;
- fire protection;
- workmanship supervision; and
- overall quality assurance.

The quality of construction must justify the design assumptions. Equally, design must acknowledge the practical constraints.

This publication is intended to assist in providing

consistently safe and economical applications of structural precast concrete, and at the same time allow innovation in design and construction to continue.

1.6 References

- 1.1 Park, R. "Review of Code Developments for Earthquake Resistant Design of Concrete Structures in New Zealand", *Bulletin of New Zealand National Society for Earthquake Engineering*, Vol. 14, No. 4, December 1981, pp. 177-208.
- 1.2 *Code of Practice for General Structural Design and Design Loadings for Buildings, NZS 4203:1992*, Standards Association of New Zealand, Wellington, 1992, 134 pp.
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Chapter 2

Floor Unit Support and Continuity

2.1 Introduction

The use of precast concrete flooring units is a popular and economical method of construction in New Zealand. General design and construction requirements for these units are covered in the appropriate New Zealand codes of practice [2.1, 2.2, 2.3]. More detailed design requirements, which are often used by manufacturers of the precast units, are available from overseas sources [2.4, 2.5, 2.6, 2.7, 2.22].

New Zealand Standards (and some overseas standards) now recognise the widespread use of precast concrete construction, and include specific reference to precast concrete support details. Research in this area is expanding however; new issues and newly recommended support details continue to simplify the design and construction of precast concrete floor systems.

The emphasis with precast concrete design must always be on constructability, with components detailed to reflect the sensitivity of supports to tolerances and to the seismic performance of the primary structure. This chapter seeks to provide guidance to designers, manufacturers and constructors.

2.2 Types of Support for Precast Concrete Floor Units

2.2.1 General

Support for precast concrete floor units may be simple or continuous. Both have their advantages in differing applications.

Simple support suits long spans, or heavily loaded structures, where it would be difficult and costly to provide the required degree of negative moment restraint at the supports.

Precast flooring support with moment fixity at the ends suits the more general commercial and residential type of construction, but requires attention to support details to ensure that the required degree of continuity can be achieved.

Between true simple support and full continuity, a designer may choose any degree of end continuity. In

making such a choice, however, the designer must be aware of the need to detail the end supports for crack control, as partial end continuity relies on the ability of the top reinforcing steel to yield over the continuous ends of the precast concrete floor unit. This yielding may occur at service loads and may lead to serviceability or durability problems in some applications.

2.2.2 Simple Support

Many designers and manufacturers assume precast concrete floor units to be simply supported. While very predictable in terms of serviceability criteria such as camber, deflection and vibration, simply supported spans have less redundancy and require more attention to the cumulative effects of construction tolerances. Seating lengths, movement at expansion or control joints, and support on ductile moment resisting frames are particularly critical for simply supported precast flooring units.

The New Zealand Concrete Structures Standard [2.1] requires minimum bearing lengths, as shown in Figure 2.1 (Fig C4.3 NZS 3101 Part 2) to maintain the structural integrity of precast flooring systems. The Standard also requires minimum longitudinal tension reinforcement across the end supports of some precast floor slabs, as discussed in Section 5.2.4.

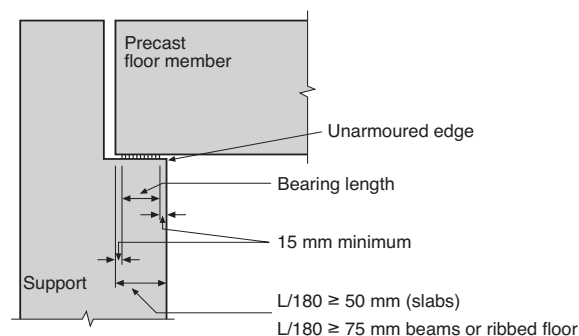


Figure 2.1: Required bearing length at the support of a member in relation to its clear span

Failures have occurred in New Zealand and overseas in large-area car parking buildings. These structures, which are exposed to daily temperature cycles and generally constructed with long clear spans, require careful attention to support details. References [2.6]

and [2.9] provide guidance on the spacing of control joints in large area structures. The primary structure must be designed to ensure that thermal movements, together with creep and shrinkage shortening, are distributed to all control joints and not concentrated at the few that offer the least resistance to movement. (Refer to Appendix C2).

Experience, both in New Zealand and overseas, shows that split or double columns are more effective than sliding bearings in providing predictable control of movement in large-area exposed structures (see Figures 2.2 and 2.3). Sliding bearings, where used, should be detailed with movement-limiting linkage bars.

Bearing pads or mortar seating pads are required to take the concentrated reactions and end rotations of all simply supported beams, or flooring units. For floor systems subjected to daily thermal movements, such as occur in the upper floor of parking structures, bearing strips that allow for differential movement between the precast floor unit and the support are essential.

Crack control in the cast-in-place topping concrete requires careful detailing if the surface will be visible in the finished structure. The placement of construction joints and saw cuts must follow the anticipated pattern of cracking, and joints should be sealed for corrosion protection where they will be exposed to the weather or to chemical attack.

2.2.3 Continuous Support

Continuity in precast concrete floors is used to:

- enhance fire resistance;
- limit deflection;
- control floor vibration;



Figure 2.2: Split column detail for control joint in large area exposed parking structure

- increase the load carrying capacity of the floor;
- provide cantilever support; and
- resist diaphragm forces.

The degree of continuity possible and its effectiveness varies with the type of precast concrete flooring units and their method of support.

Flange-supported double tees, for example, cannot easily achieve continuity, while to achieve the desired effects from continuity in other types of precast units requires consideration of support details, construction tolerances and creep and shrinkage movements in both the precast units and the topping concrete.

Flexural continuity, achieved by means of reinforcing bars placed in the topping concrete at the ends of the precast concrete flooring units, requires an adequate depth of topping concrete in which to embed the bars and maintain the minimum cover. Experience shows that 16 mm diameter bars can be effectively anchored in 65 mm thick topping, but research is required to determine safe design limits for bar size and topping thickness.

Horizontal shear between the precast member and the composite topping concrete at continuous supports can



Figure 2.3: Sliding support at a control joint that has moved more than anticipated

be adequately resisted, under static loads, by meeting the interface roughness requirements set out in the New Zealand Concrete Structures Standard [2.1]. However, recent work by Herlihy and Park [2.24] and Oliver [2.25] has shown that a more ductile shear transfer mechanism, or an alternative support mechanism, is required at the ends of precast floor slabs supported on beams that could be subject to seismic effects as discussed in Appendix A.

2.3 Precast Floor Unit Seating

Adequate support of precast concrete floor units is one of the most basic requirements for a safe structure.

The following factors must be considered when determining required seating lengths.

2.3.1 Tolerances

As discussed in Chapter 8, in the design of seating lengths allowance must be made for tolerances arising from:

- the manufacturing process;
- the erection method; and
- the accuracy of other construction.

2.3.2 Construction Methodology

If propping is to be avoided during construction, the

specified seating length must be increased to allow for the cumulative effect of the various tolerances previously mentioned. A seating length that is too short can lead to failure during construction, or a bearing failure in the completed structure. For systems that are propped during construction, the effect of an unintentionally short unit may not be serious, as cast-in-place concrete and additional support reinforcement can enable the gap to be bridged (see Figure 2.14).

2.3.3 Transverse Load Distribution

Some ribbed systems, such as double tees, have limited capacity to redistribute loads transversely in the event of damage to support concrete under a rib. As a result of this sensitivity, NZS 3101 [2.1] requires support lengths for ribbed floors to be increased by 25 mm (see Figure 2.1).

2.3.4 Volume Changes and Thermal Effects

Volume changes as a result of concrete shrinkage, creep and temperature effects, may cause axial shortening which reduces the actual seating lengths. Cracking may also result (see Figure 2.4). Reference 2.9 and Appendix C2 give appropriate methods for calculating the amount of this movement.

Thermal effects are most significant on a floor or roof exposed to the sun. Appreciable hogging due to differential temperature effects can occur. If allowance is not made for these displacements or induced actions, dam-

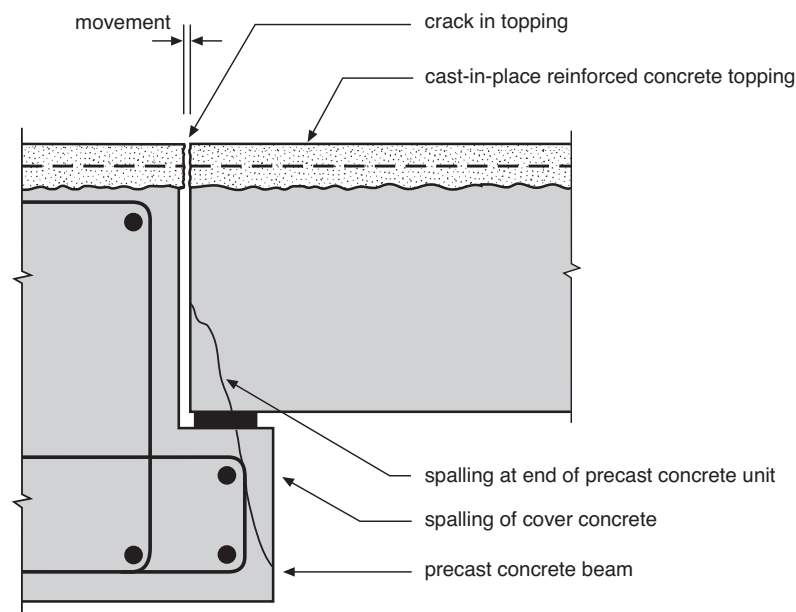


Figure 2.4: Damage of bearing seat due to movement

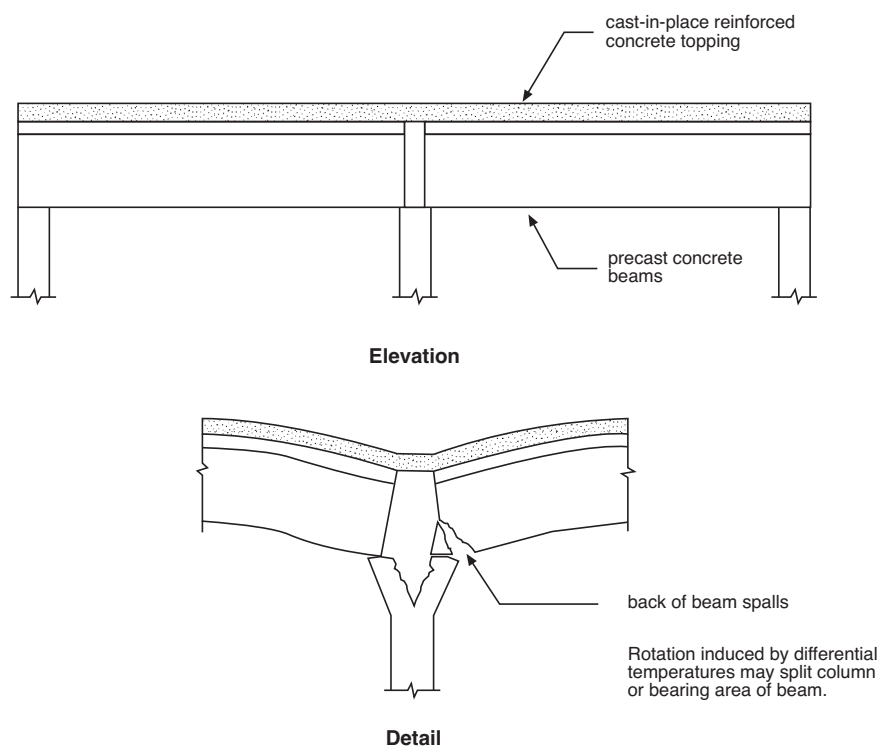


Figure 2.5: Structural damage due to differential temperature

age can occur [2.11] as illustrated in Figures 2.5 and 2.6.

Temperature gradients obtained from measurements of bridge structures may be used in design [2.12]. In New Zealand, the top levels of car parking buildings have proved particularly vulnerable to differential temperature problems due to upward camber. Bearing pads that allow slip to occur without edge spalling are recommended.

2.3.5 Seismic Effects

Seismic actions on a building can be expected to adversely affect the support conditions of precast concrete floor units in the following two ways.

(i) Loss of Beam Cover Concrete In Plastic Hinge Regions

Particular care needs to be given to rib and infill precast concrete systems due to their limited ability to redistribute loads laterally if the support to one or more ribs is lost due to the spalling of support beam cover concrete in “plastic hinge regions” (zones of plasticity in the beams). The reinforced concrete topping, which relies on tensile bond between the topping and the precast unit, cannot be expected to transfer shear force from the precast flooring to the supporting beam. In these beam regions it is recommended that concrete or steel corbels, or top flange hung details that do not rely

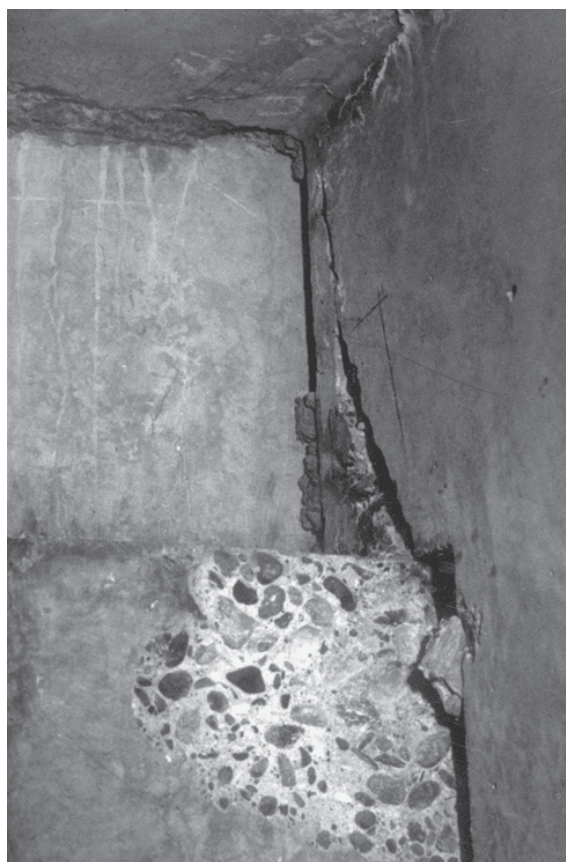


Figure 2.6: Spalling at double tee support due to differential temperature bowing of a parking structure exposed to the sun

on cover concrete, should be used unless it can be shown that transverse load shedding can occur to ribs or webs outside the plastic hinge region. Arch, or membrane, action provides an effective mechanism to transfer load to adjacent ribs where there are stirrups connecting the precast rib to the topping.

(ii) Elongation of Ductile Moment Resisting Beams

Seismic actions can cause elongation of moment resisting beams due to plastic extension of the longitudinal reinforcing steel. Refer to Appendix A1 for details.

Both (i) and (ii) above relate to structural movements associated with significant seismic damage. It is recommended that a load factor for gravity loads of not less than 1.1 be used for this design load case.

Alternatives to providing the necessary seating lengths are:

- to provide hanger bars (as shown in Figures 2.7 and 2.8) of sufficient ductility to accommodate the anticipated movement; and
- to investigate other load paths such as transverse load distribution or catenary action of topping reinforcement if the precast floor units are adequately tied into the topping concrete diaphragm by reinforcing steel detailed to accommodate the anticipated displacement.

Recent research [2.23, 2.24, 2.25] has provided simple solutions to this potential problem.



Figure 2.7: Support bars for precast concrete floor units

2.3.6 Appropriate Seating

It is essential that floor systems do not collapse as a result of any imposed movements that reduce seating lengths or cause spalling of seating. Test results [2.13] indicate that top reinforcement in slabs cannot be expected to provide an adequate load path for support forces (see Figure 2.9 (a)).

It is therefore recommended that special reinforcement be provided where the specified seating length (at either end of the precast floor slab), minus tolerances and allowances for volume changes and edge spalling, is less than the calculated increase in span due to elongation associated with plastic hinging of adjacent beams, as may occur in a beam forming part of a ductile frame during a severe earthquake (Appendix A).

Special reinforcement required to provide an effective

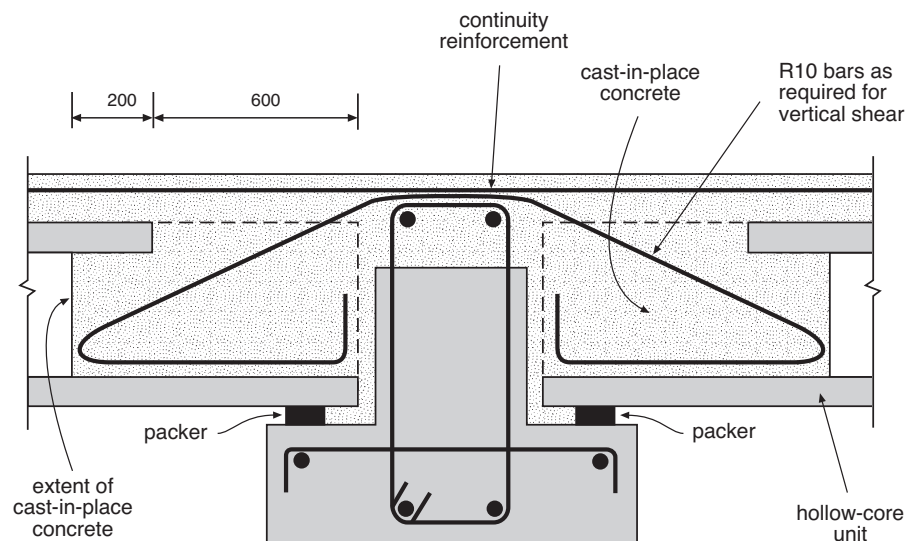
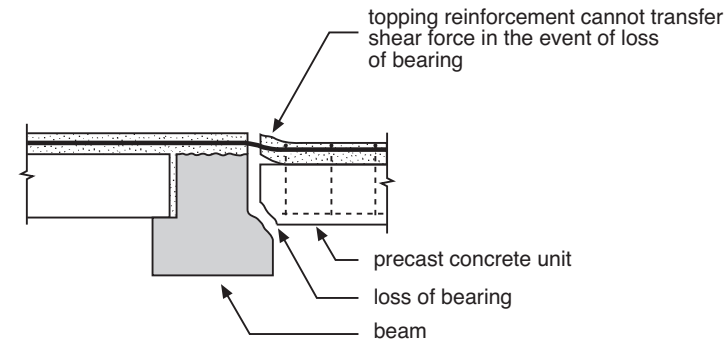
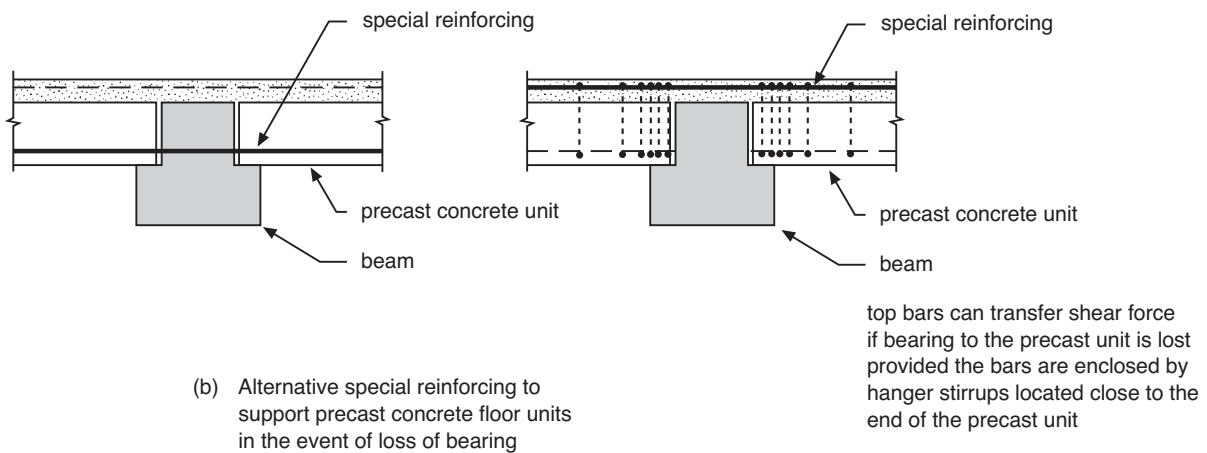


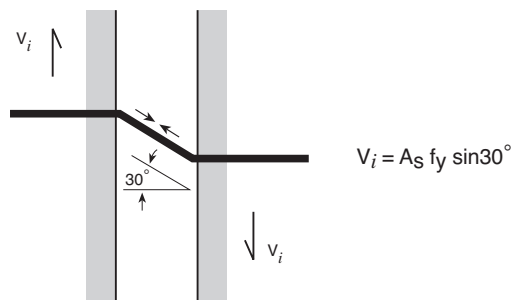
Fig 2.8: Remedial technique for lack of bearing length with Type 2 support system



(a) Inability of topping to transfer shear stress



(b) Alternative special reinforcing to support precast concrete floor units in the event of loss of bearing



(c) Kinking of the special reinforcing to provide a load path from the precast concrete floor unit to the beam

Figure 2.9: The use of special reinforcing steel to prevent collapse of precast concrete floor systems

load path by kinking in the event of loss of bearing to the precast concrete unit is illustrated in Figures 2.9 (b) and (c).

With concrete masonry construction, ribbed precast concrete flooring units should be seated on the infill concrete, not the shell of the block. Flat slab or hollow-core flooring may be seated on the block face thickness but temporary props are recommended to avoid failure during construction, if the top of the block wall has not already been fully grouted.

The use of packing or bearing strips between the precast concrete member and its support is recommended when damage to either the precast concrete unit or the support is likely to occur. Examples of where this recommendation should be followed are:

- leg or flange supported double tees;
- heavily loaded beams;
- precast units bearing on steel supports;
- uneven bearing surfaces of cast-in-place concrete or reinforced concrete masonry; and
- any precast flooring units subjected to daily temperature variations.

Bearing strips or packing may consist of the following.

- Sand cement mortar, suitably plasticised and of sufficient consistency to adequately support the precast unit without squeezing out of the joint. Shims may be required to maintain the correct joint thickness. Note that sand-cement mortar is not suitable for floor slabs subjected to daily temperature movements (such as the top level of car parking structures).
- Neoprene rubber pads.
- Proprietary plastic shims or strips (recommended for slabs subjected to daily temperature movements).
- Epoxy mortar.

2.4 Seating Details for Precast Concrete Hollow-core and Flat Slab Floor Units

The types of support for precast concrete hollow-core or flat slab flooring units seated on beams can be divided into the three groups as shown in Figure 2.10. The differences between these types are the depth of the supporting beam prior to the cast-in-place concrete being poured. Each type is examined in the following sections.

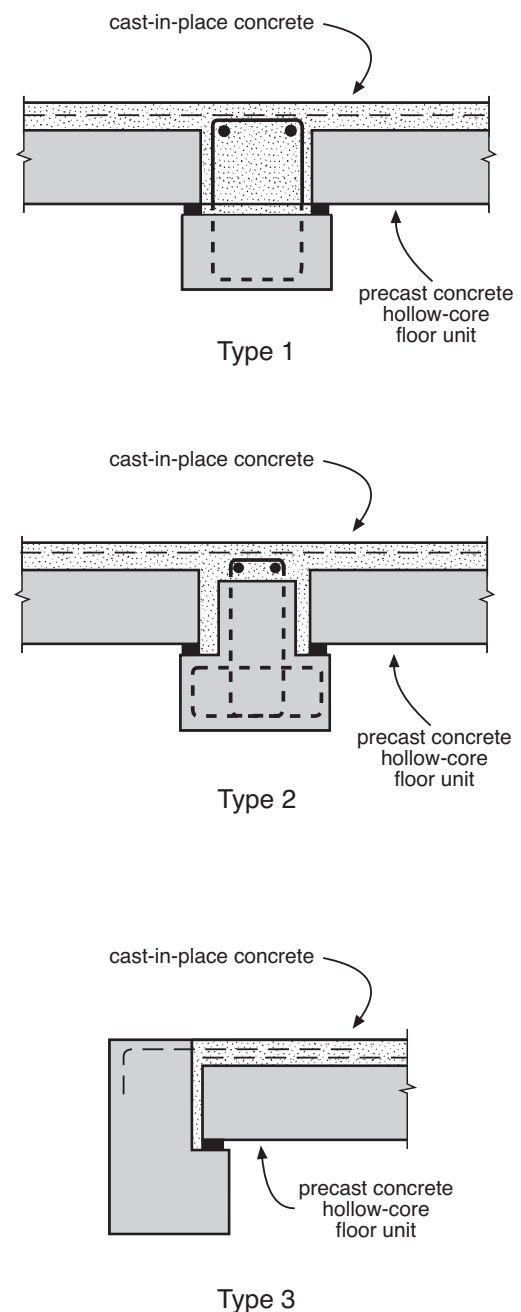


Figure 2.10: Types of support of precast concrete hollow-core floor units by precast concrete beams

2.4.1 Type 1 Support (Figures 2.11 and 2.12)

Advantages of the Type 1 Support

- As the top surface of the first stage of the beam is at the level of the soffit of the precast concrete floor unit, cast-in-place concrete can easily be placed between the floor units and into recesses at the ends of hollow-core units. This enhances the shear capacity of hollow-core units.

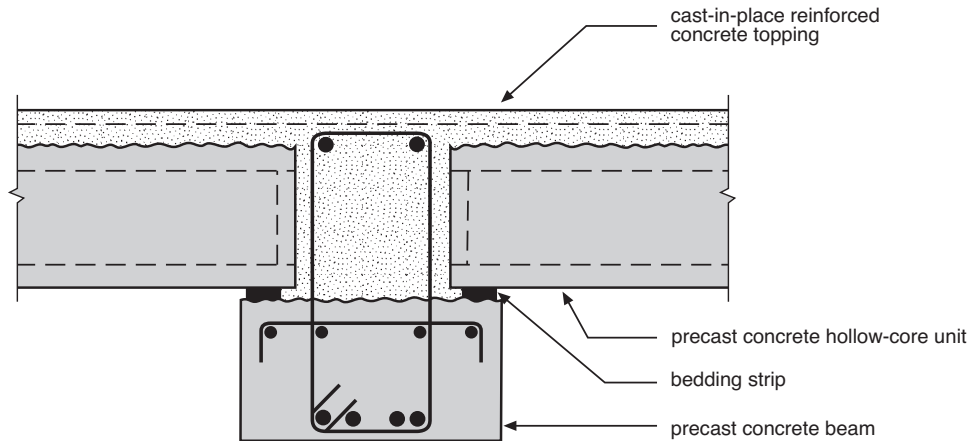


Figure 2.11: Type 1 precast beam support system

- (ii) The presence of well compacted cast-in-place concrete against the bottom of the precast concrete floor unit enables reliable negative moment continuity to be developed.
- (iii) Placement of reinforcement to provide ductility and load capacity following loss of slab support due to seismic effects is simplified (Figure 2.13).
- (iv) With the Type 1 method of support for hollow-core units, problems of construction tolerances can be easily overcome on site. The method illustrated in Figure 2.14 has been successfully used for length discrepancies of up to 100 mm with the detail as constructed being checked by load tests as required by NZS 3101. Studies have confirmed the adequacy of this method [2.14, 2.21, 2.24]. Provided that extra topping steel (saddle bars) plus steel “paper clip” bars in the broken-back voids are used, the shear strength is similar to the case with a 40 mm seating length and no special detailing. In these tests failure of the precast concrete units, which was initiated by debonding of the prestressing

strands at the ends of the units, occurred at an acceptable load level.

Disadvantages of the Type 1 Support

- (i) Due to the reduced depth of beams at the stage when precast floor units are erected, more propping is generally required than with other support types. If propping is not desirable, the adequacy of the strength and stiffness of the beam during the construction phase must be considered by the designer.
- (ii) Additional reinforcing steel may be required at the top of precast beam units to provide negative moment capacity over the supporting props during construction.
- (iii) Additional negative moment reinforcement will be required to resist the dead load continuity moments at the beam supports after the props have been removed (based on the full-depth composite beam). This additional beam strength may require an increase in the column strength to ensure that beam hinging is the ductile mechanism for seismic resistance.



Figure 2.12: Example of a Type 1 precast beam support system

2.4.2 Type 2 Support

Advantage of Type 2 Support

- (i) With this form of support (See Figures 2.15 and 2.16), the precast concrete beam depth may extend to the top of the floor unit. Because the beam depth is greater when the floor units are erected, less propping is required.

Disadvantage of Type 2 Support

- (i) If the vertical gaps between the precast beam and floor units are too small, there may be difficulty in

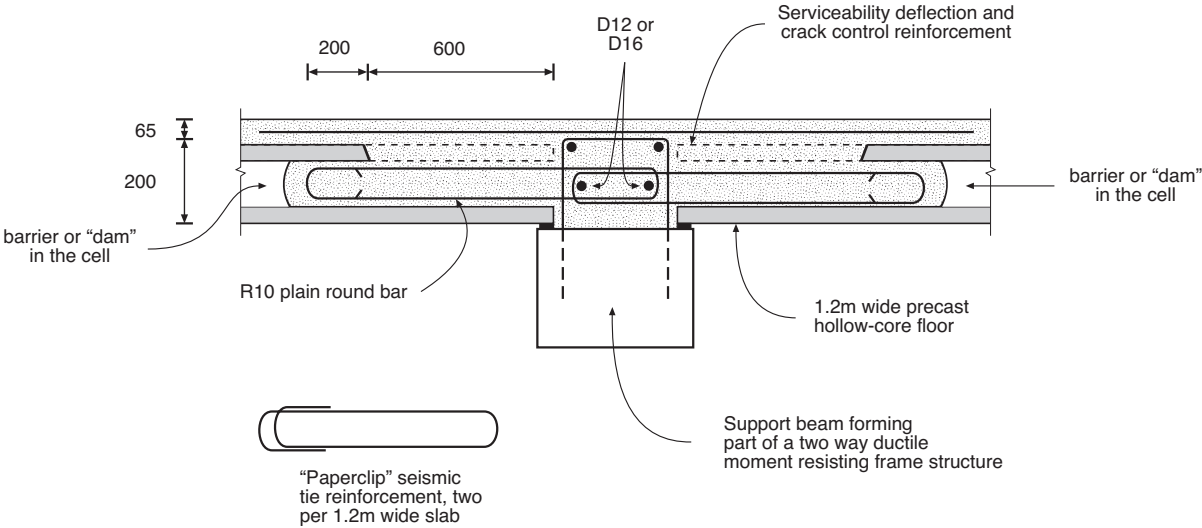


Figure 2.13: Recommended final tie detail for hollowcore slabs in ductile moment-resisting frame construction. Herlihy and Park [2.24]

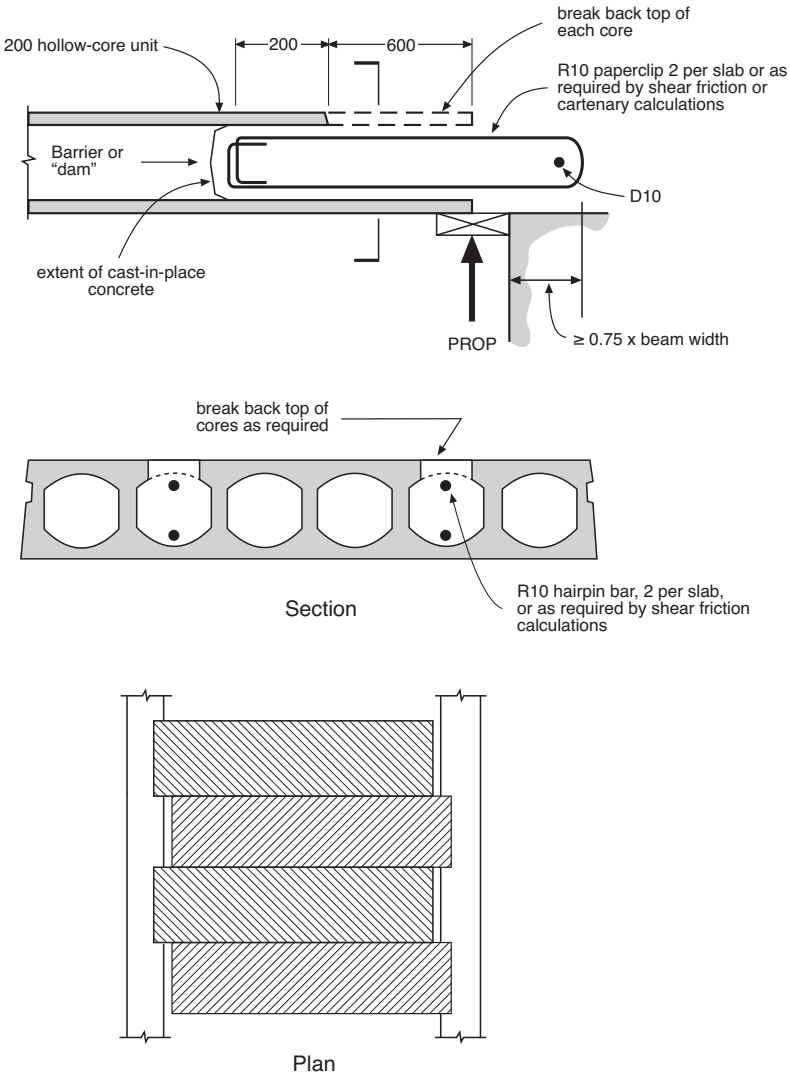


Figure 2.14: End support detail when seating is inadequate [2.26]

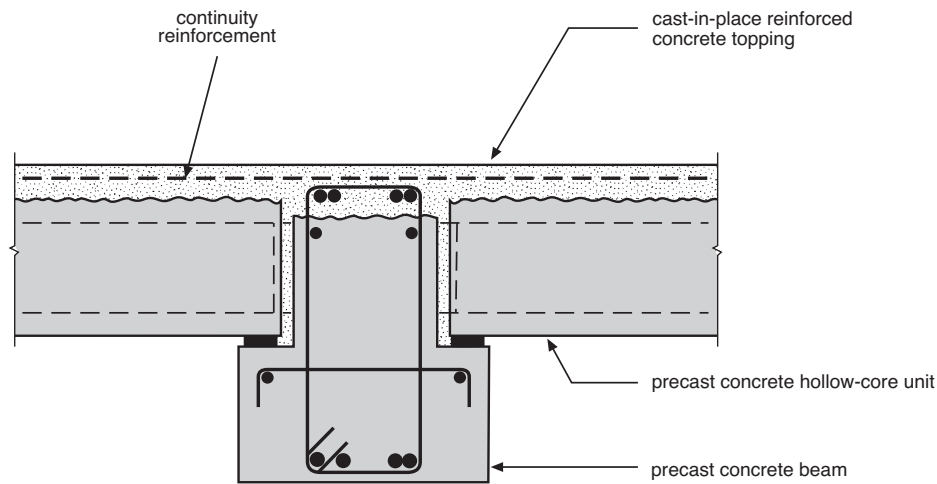


Figure 2.15: Type 2 precast beam support system

placing concrete both in the gap and in the recesses of hollow-core units. This can reduce the shear strength of the precast flooring and prevent the development of negative bending moment actions in the floor units resulting in a detrimental influence on serviceability. If this system is to work as intended, the precast flooring units must be suitably detailed. It is recommended that some of the cores of hollow-core flooring units are broken out to enhance the shear strength.

- (ii) There is a construction difficulty if the accumulated tolerances reduce the actual bearing length at the ends of precast concrete floor units. The site remedial method, which is suitable for Type 1, is not possible. An alternative shown in Figure 2.8 could be adopted. Adequate ductility in the tie bar detail is required for floor slabs seated on beams that could be forced apart by seismic actions as described in Appendix A1 [2.23]. This detail is

similar to one that is recommended by the FIP [2.5]. The bar size is designed for the ultimate shear load.

- (iii) Beam stirrups are anchored in the thin, cast-in-place concrete topping. Horizontal shear stresses due to flexure must be transferred from the top of the precast beam into the topping concrete. Some degree of anchorage of stirrups is also necessary. A relationship between stirrup diameter and minimum topping thickness has been suggested [2.15]. However, the recommendations were based on monotonic loading tests and may not be conservative for plastic hinge regions in seismic resisting beams. There is need for further research.

2.4.3 Type 3 Support

Figures 2.17 and 2.18 show this support system, which may be used for perimeter beams or lift and stairwells. No edge formwork for slab topping concrete is required. As for Type 2, it is recommended that the cores of hollow-core flooring units be broken out to ensure adequate compaction of cast-in-place concrete. For this reason, even more attention must be paid to providing adequate seating. If, for example, a short unit is not to be rejected, some type of structural steel bracket, such as that shown in Figure 2.19, is required.



Figure 2.16: Example of a Type 2 precast beam support system

2.5 Types of Support for Ribbed Units

In general, with the exception of cantilever support, it is unrealistic to assume that moment continuity for serviceability limit states can be achieved at the supports of ribbed flooring units. Combined creep and shrinkage shortening in the highly prestressed rib can

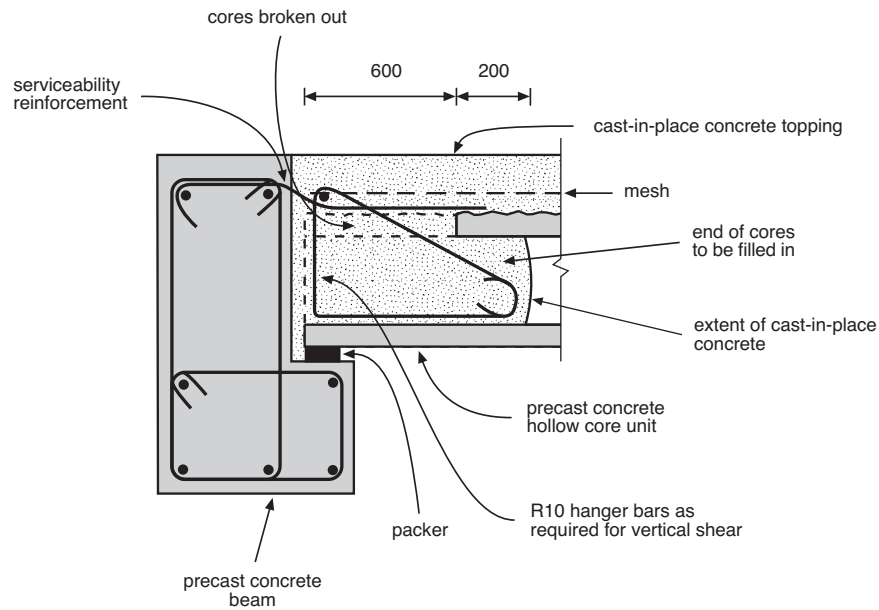


Fig 2.17: Type 3 precast concrete beam support system

form a gap between the end of the rib and the cast-in-place beam concrete. Epoxy mortar and epoxy injection have been attempted, but because the operation cannot be inspected properly such practice is not recommended [2.16]. The same comments apply to the use of dry pack mortar: it is difficult to use and hard to verify correct placement.

When the beam structural depth is to be minimised, precast concrete units may be supported by their flanges. Reference 2.6 provides two design methods. Designers should appreciate the sensitivity of the strength of flange hung details to tolerances. Allowance for this and other influences affecting precast concrete floor unit seating are discussed in Chapter 8.

2.6 Special Diaphragm Requirements for Hollow-core Floors

The New Zealand Concrete Structures Standard [2.1] allows reduced roughness for the top surface of machine produced extruded hollow-core floor slabs as compared to other precast concrete floor units.

There is convincing evidence [2.19] that a 5 mm amplitude roughness at the interface of precast extruded hollow-core slabs and cast-in-place topping concrete is unnecessarily conservative for flexural shear transfer. This is because the dry concrete mixes and normal curing regimes used for the production of

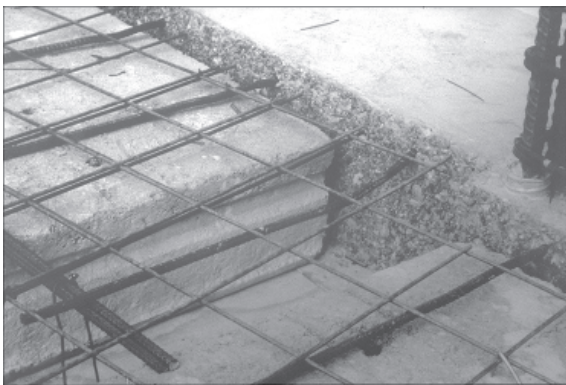


Fig 2.18: Example of a Type 3 precast concrete beam support system

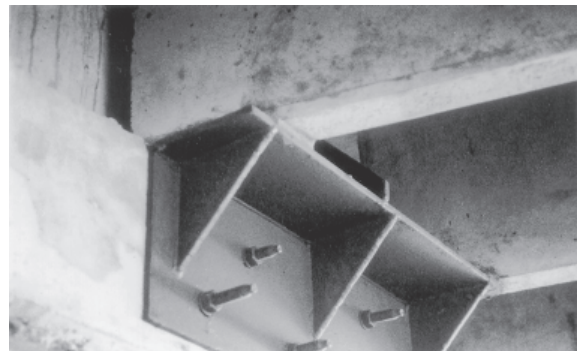


Fig 2.19 : Steel fixing providing support to double-tee webs

extruded hollow-core slabs do not cause an accumulation of laitance on the top surface of the slabs. A light brushing (similar to the U4 class of finish shown in Figure 14 of NZS 3114: 1987 [2.20]) has proven to be adequate to ensure composite action under gravity loads. These comments apply to dry mix extruded concrete slabs, but not to conventionally cast flooring units.

Recent research [2.24] has found that for hollow-core floor slabs subjected to direct tension forces from beam elongation, as described in Appendix A, the bond between topping concrete and the top surface of precast hollow-core floor slabs cannot be relied on to transmit diaphragm forces to the seismic resisting frames or walls. The details shown in Figures 2.20 and 2.21 are recommended as a means of transferring diaphragm forces.

Shear ties, when required to satisfy code requirements [2.1], can be placed in the shear keys between adjacent hollow-core units or in holes cut into selected cores. Where the hollow-core flooring units are supported on beams that form part of a ductile moment-resisting frame, beam elongation (as described in Appendix A) may cause separation of the topping from some of the hollow-core units. Shear ties anchored in the slab cores will provide a more dependable connection in this case.

Calculations on the horizontal shear capacity of topped extruded hollow-core floor diaphragms usually ignore the capacity of the hollow-core units and take all the diaphragm shear on the topping concrete. Further research on the shear capacity of topped and untopped hollow-core floor diaphragms is required before a less conservative approach can be taken.

2.7 Overseas Practice

The Prestressed Concrete Institute (PCI) [2.4] has produced details of connections for hollow-core units. In most cases there is no structural topping shown in the PCI details, nor any flexural continuity provided at the ends of units. There also appears to be a preference for

full-depth precast beams. Horizontal and draped reinforcement is used to provide continuity for diaphragm action and it is grouted in the gaps between units as shown in Figure 2.20.

European practice is similar [2.5, 2.22]. The steel is anchored either into the cores, which have been broken out at the top, or the joints (see Figure 2.21).

In both the PCI and European details, anchorage of critical tie reinforcing bars in the joint between slabs may create a potential problem in structural frames that could undergo significant beam elongation due to the formation of plastic hinges, as outlined in Appendix A. Tie bars anchored in the cores of hollow-core slabs (lapped with the pretensioning tendons in the slabs) appear to offer a more dependable load path [2.24, 2.25].

2.8 Related Considerations

2.8.1 Fire Resistance

Manufacturers of precast concrete flooring components can produce units with a Restrained or an Unrestrained Fire Resistance rating [2.17]. Floor slab manufacturer's product literature should indicate the method of achieving the required fire resistance. In general terms (for normal weight concrete), the lighter the self weight of a flooring system, the more it relies on restraint to achieve fire resistance. The designer using a Restrained Fire-rated floor system must check that the structure can provide the required degree of restraint.

Restraint may be provided by end moment continuity, resistance to thermal expansion, or a combination of both of these effects. Reference 2.17 sets out the methods of calculating the required degree of restraint. The ability of the structure, and the floor unit support, to provide this restraint must then be checked. Exterior bays, with moment continuity only possible at one end and with limited resistance to thermal expansion, are particularly critical.

The use of heavier precast concrete units with Unre-

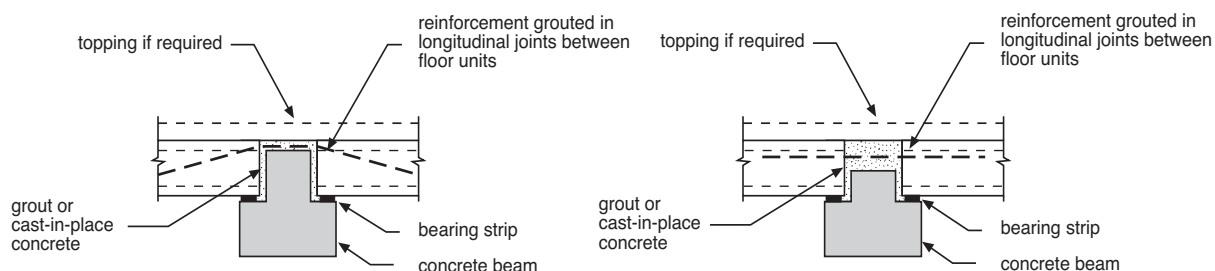


Figure 2.20: Details of continuity steel (from Reference 2.4)

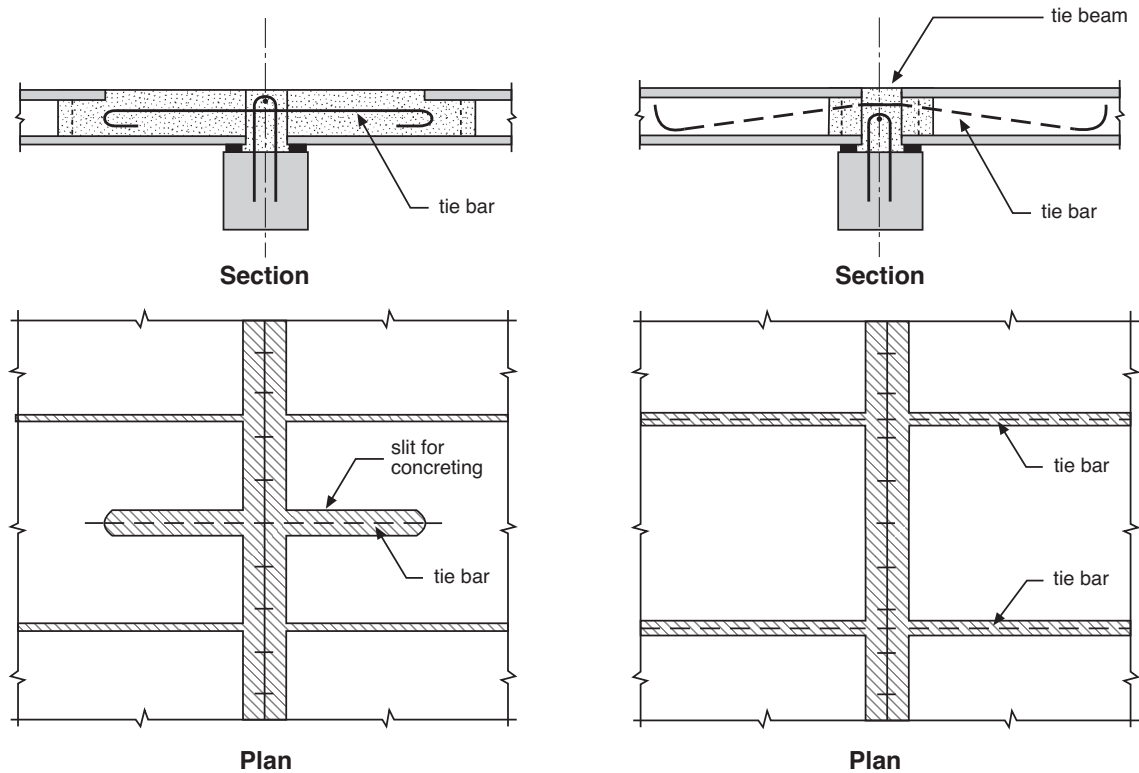


Figure 2.21: Provision of continuity or tie steel (from Reference 2.5)

strained Fire Resistance ratings avoids the problems of restraint in the exterior bays of a structure. Flange-supported double tees must also be designed as unrestrained units (in fire engineering terms) as the thermally induced forces located above the centroid of the concrete section can significantly reduce the fire resistance rating of the floor system.

2.8.2 Deflection and Vibration Control

The use of structural continuity at the supports of precast concrete floor units requires judgment and caution, at serviceability limit states. Highly prestressed narrow webbed units, such as leg supported double tees and ribbed multi-piece floors, can undergo appreciable creep shortening that removes the end bearing essential for continuity at service loads. This has more effect on service load deflection than on vibration, possibly due to sliding friction at the ends of the unit providing damping.

Dry mix extruded floor slabs are not normally as highly stressed and so do not creep or shrink as much as conventionally cast ribbed units. Moment continuity is therefore more effective for the control of deflection at service loads in extruded hollow-core slabs.

Vibration in precast concrete floor systems has not been a serious problem in New Zealand. As spans are extended however, and as manufacturers respond to

designers' needs for lighter weight floors, vibration characteristics need to be checked.

The critical parameters for human perception are frequency, amplitude and damping. Reference 2.18 provides guidance on recommended values. Typical office construction without partitions can be expected to provide 3% to 4% of critical damping.

For commercial use, floor frequencies greater than 6 Hz are recommended while for gymnasiums or areas used for aerobic exercises frequencies of 2.5 Hz or multiples of 2.5 Hz should be avoided.

2.9 Recommendations

- Precast concrete floor unit support details must reflect practical achievable tolerances and the anticipated seismic performance of the supporting structure (Sections 2.3.1 and 2.3.6).
- Simply supported units require specifically detailed design at movement control joints (Section 2.2.2).
- Moment continuity support requires consideration of creep and shrinkage. Topping thickness must be adequate to provide reinforcement embedment for continuity (Section 2.2).

- Precast concrete floor unit seating should follow the manufacturer's recommendations. If reduced seating lengths are required, the manufacturers of the precast components and the constructor of the building must take special precautions to ensure safety during construction. Reduced seating may also require additional reinforcement to prevent collapse in the event of gross seismic damage to the primary structure (Section 2.3).
- Suitable bearing material is required to prevent concrete spalling where precast concrete units seat on rigid supports (Section 2.3.7) or where daily temperature movements occur (Sections 2.2.2 and 2.3.4).
- Type 2 or Type 3 supported hollow-core or flat slab floors require special attention to tolerance to ensure adequate end support and moment continuity.
- Support for ribbed units requiring moment continuity must be detailed to allow for construction tolerances, creep and shrinkage effects, and to ensure the ease of placement of well consolidated concrete in critical parts of the support.
- If moment continuity at supports is used to limit deflection or to reduce the human perception of vibration, special calculations are required (Sections 2.8.1 and 2.8.2).

2.10 References

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Chapter 3

Frame Connections

3.1 Introduction

Experience of earthquakes, and extensive laboratory testing, have shown that well-designed, detailed and constructed cast-in-place continuous reinforced concrete frames perform very well during severe earthquakes. Moment-resisting frames incorporating precast concrete members, designed to be ductile and providing the primary earthquake resistance, have not had the same extensive laboratory testing. The use of precast concrete in moment-resisting frames was shunned for many years in New Zealand, due mainly to the observation of poor performance of connection details between the precast elements during major earthquakes in many overseas countries. However, moment-resisting frames incorporating precast concrete members have become widely used in New Zealand since the 1980s [3.1].

Confidence in the use of precast concrete elements in moment-resisting frames in New Zealand has required the use of capacity design to ensure that yielding during a major earthquake occurs only in the preferred ductile regions of the frame. Also, moment-resisting frames containing precast concrete elements have been designed and constructed so as to possess stiffness, strength and ductility similar to that of cast-in-place concrete monolithic construction. In other words, monolithic construction is emulated [3.1].

The basic challenge in the design of building structures incorporating precast concrete elements for earthquake resistance is in finding an economical and practical method for connecting the precast elements together so that the seismic performance will be as for a monolithic structure. If the connections between the precast elements are placed in critical regions, such as in potential plastic hinge zones, the design approach is to ensure that the behaviour of the connection region approaches that of a monolithic cast-in-place structure. Possible brittle connections between members should be made overstrong and placed away from the critical regions. Reinforcing details and structural configurations should be arranged to ensure that potential plastic hinge regions are away from the jointing faces of precast members if possible [3.1].

The general trend in New Zealand for reinforced concrete framed buildings incorporating precast concrete is to design the perimeter frames with sufficient stiff-

ness and strength to resist most, if not all, of the seismic loading. The interior columns of the building then carry mainly gravity loading and can be more widely spaced. References 3.2 to 3.8 give details of several buildings designed in New Zealand since the 1980s, which incorporate significant quantities of precast concrete in their frames and floors.

The New Zealand standards for concrete design current in the 1980s [3.9, 3.10], like the design standards of many countries, contained comprehensive design provisions for the seismic design of cast-in-place concrete structures, but did not have seismic provisions covering all aspects of precast concrete structures. The revisions of these standards published in the 1990s [3.11, 3.12] contain more design provisions for the seismic design of structures incorporating precast concrete as a result of the significant research and development conducted in New Zealand during that decade.

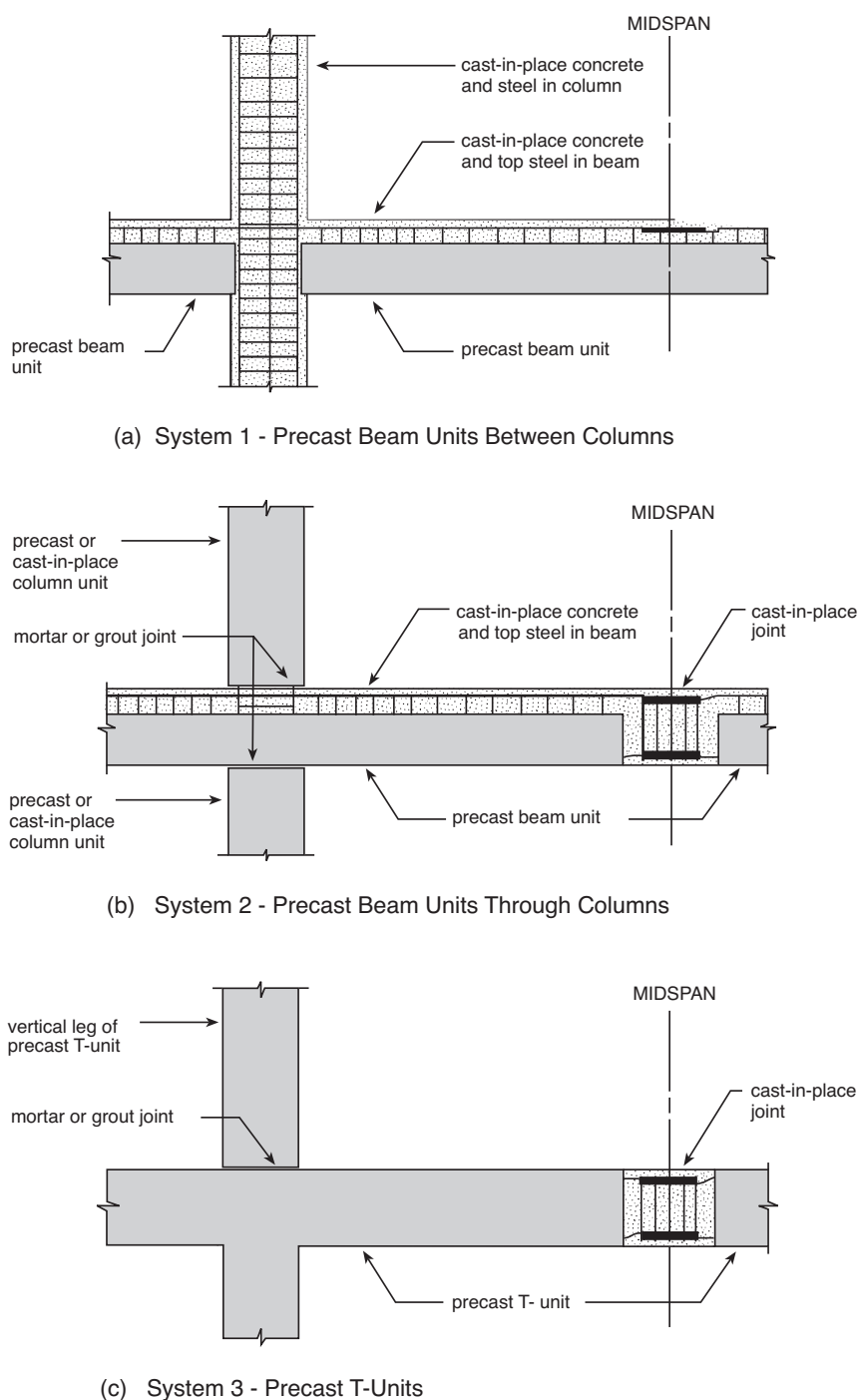
A number of possible arrangements of precast concrete members and cast-in-place concrete forming ductile moment-resisting multi-storey reinforced concrete frames, commonly used for strong column-weak beam designs, have been identified [3.1] and are shown in Figure 3.1. The three systems illustrated are described below. These systems can also be used in a modified form for one- or two-storey frames where strong beam-weak column design is permitted. The aim has been to design the systems so to achieve behaviour as for a monolithic structure. Ductile frames are designed using the capacity design procedure and according to the provisions for totally cast-in-place concrete structures, or alternatively the frames can be designed using the limited ductility procedure [3.11, 3.12].

This chapter discusses the possible arrangements of precast members in moment-resisting frames, comments on some design aspects and test results, and concludes with recommendations.

3.2 System 1 - Precast Beam Units between Columns

3.2.1 Construction Details

An arrangement involving the use of precast members to form the lower part of the beams is shown in Figure 3.1(a). The precast beam elements are placed between



Notes:

- 1 Shading denotes precast concrete
- 2 Reinforcement in the precast concrete is not shown
- 3 Capacity design is used to ensure that the flexural strength of the columns is suitably greater than the flexural strength of the beams, so that in the event of a severe earthquake plastic hinging occurs in the beams rather than in the columns.

Figure 3.1: Arrangements of precast members and cast-in-place concrete for constructing moment-resisting reinforced concrete frames [3.1]

columns and seated on the cover concrete of the previously cast-in-place reinforced concrete column below and/or propped adjacent to the columns. In some cases there may be two precast beam elements per span, requiring additional props, with a cast-in-place joint at midspan where longitudinal beam bars are spliced. A precast concrete floor system is placed, seated on the top of the precast beam elements and spanning between them. Reinforcement is then placed in the top of the beams, over the precast floor and in the beam-column joint cores. The topping slab over the floor system and the beam-column joint cores are cast. The next storey height of columns is then prepared.

3.2.2 Some Design Aspects

Anchorage of Bottom Longitudinal Bars

A possible difficulty with this connection detail is that the bottom longitudinal bars of the beams, protruding from the precast beam elements, need to be anchored in the joint cores. Hence the column dimensions need to be reasonably large to accommodate the required development length and to reduce the congestion caused by the hooked anchorages.

The previous concrete design standard [3.10] required that beam bars that are terminated at an interior column should be passed right through the core of the column and be terminated with a standard hook immediately outside the ties around the perimeter of the column core.

However, full-scale laboratory cyclic load tests in New Zealand [3.13 - 3.16], showed that anchoring all the bottom bars within the joint core with a hooked lap did

not affect the seismic performance of the joint. In the specimens tested all the bottom flexural steel was terminated in standard 90° hooks in the far side of the joint core.

Hence in the 1995 revision of the concrete design standard NZS3101:1995 [3.12], this design provision was amended to permit the anchorage detail within the joint core of interior columns shown in Figure 3.2.

The anchorage of beam bars within the joint core in System 1 can then be designed using the same code rules as for anchorage in an exterior column. The anchorage is considered to commence at one-half of the depth of the column or $8d_b$ from the face at which the bar enters the column, whichever is less, where d_b is the bar diameter.

Location of the Cast-in-place Concrete Precast-concrete Cold Joint

A further possible problem is that the critical section of the potential plastic hinge region in the beam occurs at the vertical cold joint at the column face between the cast-in-place concrete of the joint core and the precast beam. Figures 3.3 and 3.4 show examples of this type of construction using System 1 (in Figure 3.1).

In some of the full-scale laboratory tests [3.10, 3.11], it was found that by the end of the tests vertical sliding shear displacements were occurring at the cold joints at the column faces. In another test [3.15], no movements occurred at the cold faces at the ends if the precast beams were seated on 30 mm of cover concrete of the column below. Tolerances in normal construction may mean that this seating is reduced. Hence it is recommended

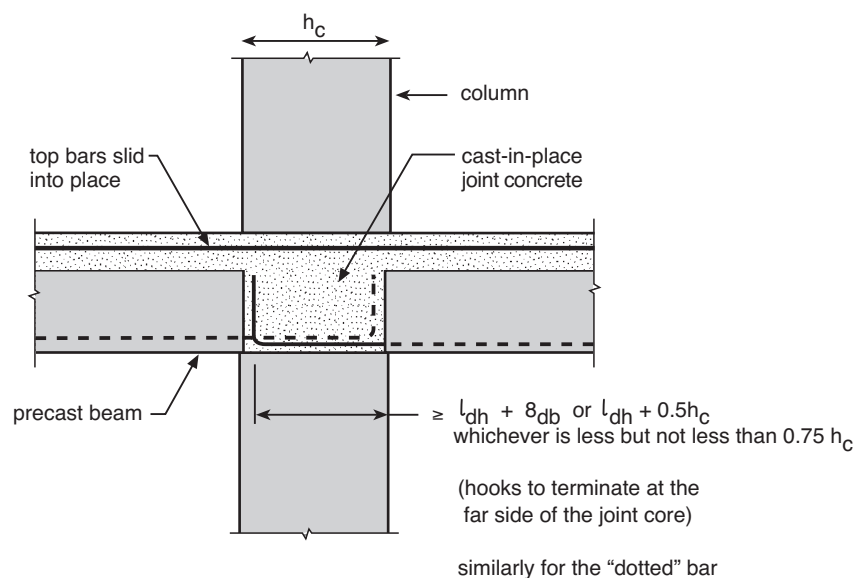


Figure 3.2: Hooked lap of bottom bars within joint core for System 1

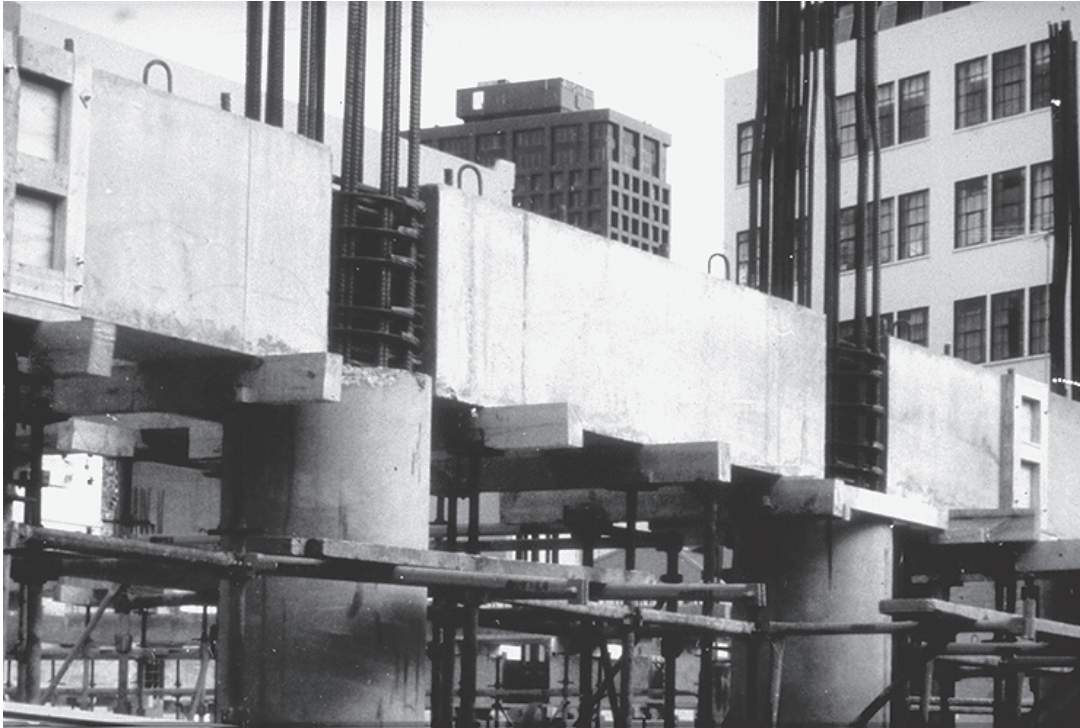


Figure 3.3: Frame incorporating precast elements in the beams between columns (System 1)

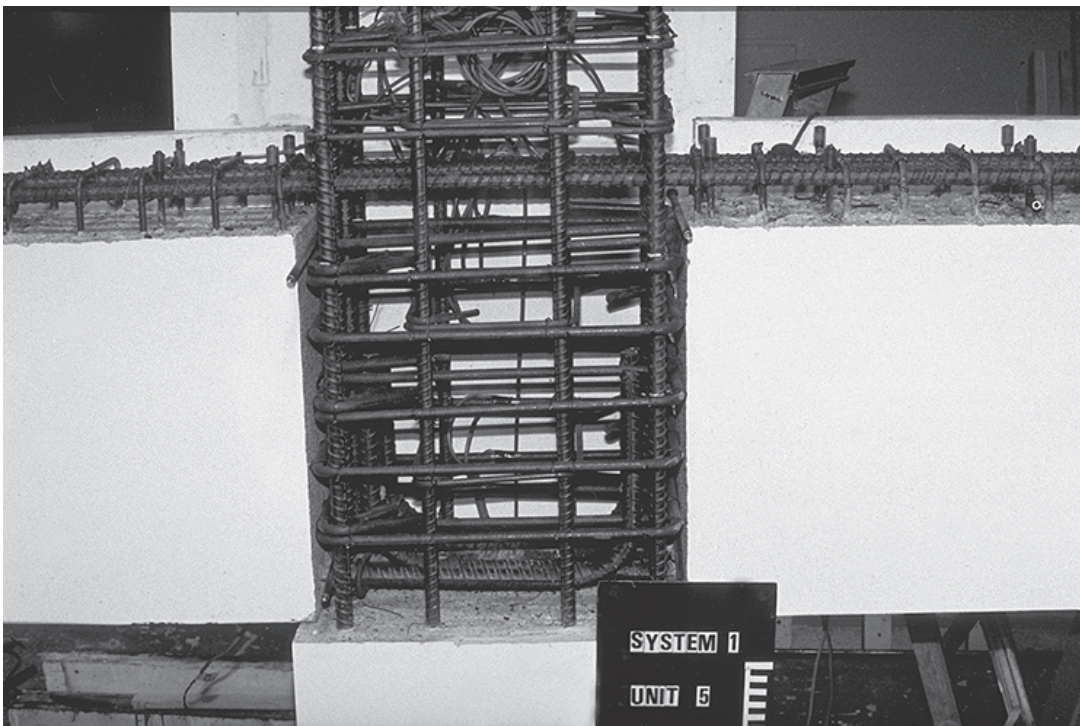


Figure 3.4: A detail of the joint region for a System 1 frame [3.15]

[3.12] that vertical shear should be transferred across these interfaces by shear friction or by mechanical keys. That is, the end of the precast beam should be clean, free of laitance and intentionally roughened to a full amplitude of not less than 5 mm. Alternatively, the key could take the form of either reinforced concrete projections from the end of the precast beam unit into the cast-in-place concrete of the joint core, or otherwise recesses in the end of the beam into which the cast-in-place joint core concrete could project.

The option of using such a mechanical key would make this aspect of New Zealand practice similar to overseas practice. For example, Figure 3.5 shows joint details from China, Japan and Romania where, in each case, shear keys have been provided at the vertical interfaces between the precast and cast-in-place concrete. Nevertheless, intentional roughness as described above should also provide adequate shear transfer.

Horizontal Interface between the Precast Beam Unit and the Topping Concrete

It is recommended that the top surface of the precast beam unit should be clean, free of laitance and intentionally roughened to a full amplitude of not less than 5 mm or mechanically keyed.

Reduction in Design Negative Moments

When using composite beams jointed at column locations, advantage can be taken of the presence of dead load during construction to reduce demand for negative moment reinforcement. Significant savings can be made by this design process especially in relation to joint core shear reinforcement. A description of this technique, design issues and details are given in Appendix B1.

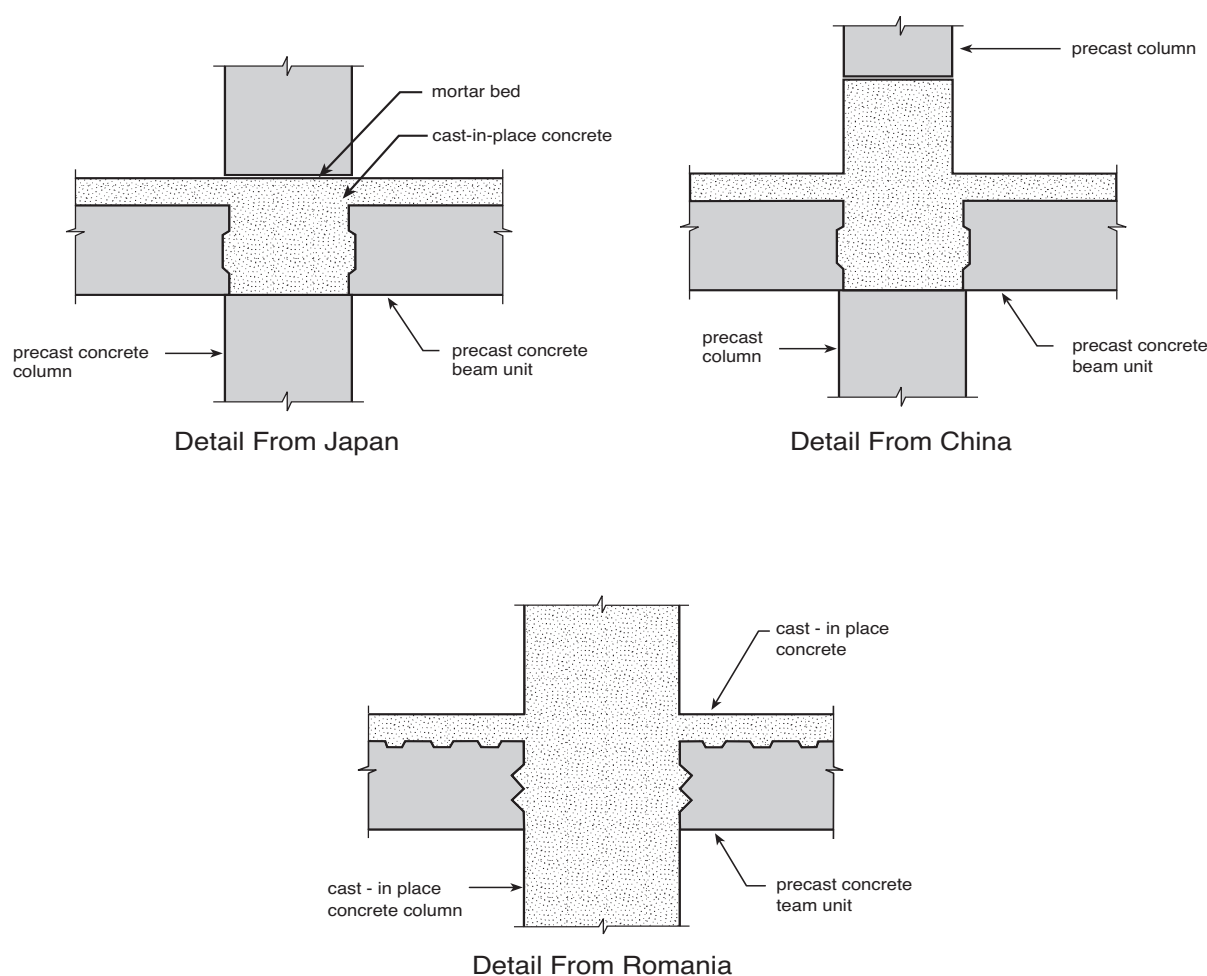


Figure 3.5: Mechanical keys used at the vertical interface between the precast and cast-in-place concrete

3.3 System 2 - Precast Beam Units through Columns

3.3.1 Construction Details

An arrangement that makes more extensive use of precast concrete, and avoids the placing of cast-in-place concrete in the congested beam-column joint core regions, is shown in Figure 3.1(b). The success of this system depends on tighter than normal tolerances. The reinforced concrete columns can be either precast or cast-in-place to occupy the clear height between beams. The precast portions of the beams extend from near midspan to midspan, and hence include within the precast element over the columns the complex arrangement of joint core hoop reinforcement, which is fabricated in the precast factory. The precast portions of the beams are seated on the concrete column below with a suitable jointing material between and propped for construction stability.

Protruding longitudinal column bars from the reinforced concrete column below pass through preformed vertical holes in the precast beam element and extend above the top surface of the element. The holes in the precast beam elements are preformed using corrugated

steel ducting and are grouted with the horizontal interface gap, as discussed in Section 3.6, after the column bars have been passed through.

Protruding bottom bars of the precast beam elements are lapped in the cast-in-place joint at midspan or, alternatively, they can be connected by welding to steel plates that are bolted together in the cast-in-place joint. A precast floor system is seated on top of the precast beam elements and spanning between them. Reinforcement is then placed in the top of the beam and topping slab, and cast-in-place concrete is poured.

One variation on this system is for the top steel of the beam to be cast within the precast beam section. This is particularly suitable for perimeter beams as no edge formwork for the topping is then required. Columns of the next storey are then positioned above the beams using grouted steel sleeves to connect the vertical bars if columns are precast, or using normal reinforced concrete details if columns are cast-in-place.

Figure 3.6(a) shows a 22-storey building under construction using this system. The structure consists of moment-resisting perimeter frames with interior frames carrying mainly gravity loading. Some construction details are shown in Figures 3.6(b), (c) and (d).



Figure 3.6(a): Structure of building frame incorporating precast concrete beam elements passing through the columns



Figure 3.6(b): Precast concrete beam corner unit being lowered into place using temporary plastic tubes as guides

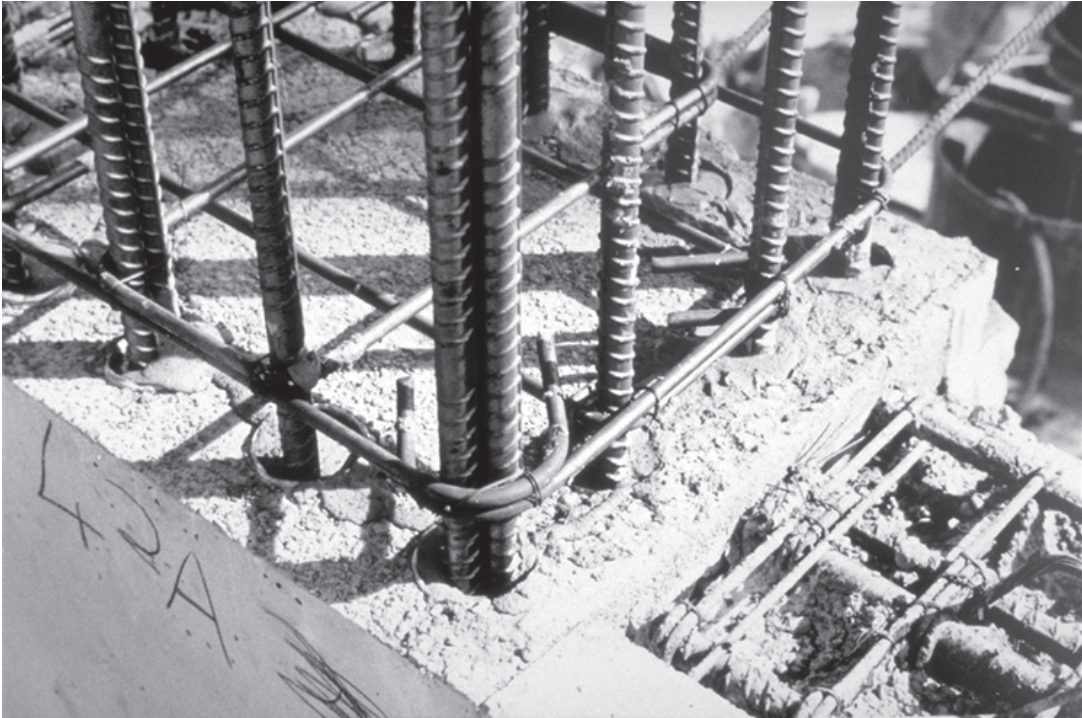


Figure 3.6(c): Column bars after being grouted in the joint core of a precast concrete beam unit

3.3.2 Some Design Aspects

General

An advantage of System 2 is that the beam-column joint core reinforcement can be incorporated in the

precast concrete beam element and the potential plastic hinge regions in the beam occur within the precast elements away from the vertical cold joints between precast elements and cast-in-place concrete. Also, it is possible to incorporate in precast concrete beam ele-



Figure 3.6(d): Preparing a cast-in-place concrete midspan joint of a perimeter frame

ments reinforcing details to relocate the potential plastic hinge regions away from the column faces if necessary (see Figure 3.7).

Splicing at Midspan of Beams

An aspect of the previous concrete design code [3.10] that caused problems in design was the requirement that, when the critical section of a potential plastic hinge region of a beam is located at a column face, no part of the splice of the longitudinal reinforcement was to occur within $2d$ of the column face, where d is the effective beam depth. To satisfy this requirement the clear span of the beam had to be at least $4d + l_s$ where l_s is the splice length. This code requirement made it difficult to use beams of relatively short span. However, as reported in Appendix B2, a number of tests on midspan joints have now been conducted [3.14, 3.15, 3.16] and the 1995 edition of the concrete design standard [3.12] modified this requirement by permitting the splice to commence at distance d from the column face. Hence conventional straight bar splices are now possible at midspan of short span beams.

Some details for midspan connections in beams which have been used are illustrated in Figure 3.8. The conventional straight bar lap of Figure 3.8(a) can be shortened by using hooked laps as shown in Figure 3.8(b) and (c). The double hooked lap (see Figure 3.8(c)), involving the use of hooked “drop in” bars, is the most convenient hooked lap to construct. These details, in

some cases with slight modification, have all shown excellent performance in laboratory tests (see laboratory tests [3.14, 3.15, 3.16] described in Appendix B2) and hence can be recommended as suitable for use.

Diagonal reinforcement has been used where shear forces in the beams are high (see Figure 3.8(d)). The design and detailing of the welded connection details require extreme care. Significant vertical ties are required between the bends in the diagonal reinforcement to resist the vertical component of the force in the diagonal bars. Also, it should be checked that bearing failure of the concrete cannot occur under the bends of the diagonal reinforcement (see the laboratory tests [3.15, 3.16] described in Appendix B2). The joints should be capacity designed and any eccentricities of plate and reinforcing bars be minimized and provided for by basketing reinforcement. Grinding back of the reinforcing bars to provide 45° double-V butt welds may be required to provide the necessary high standard welded connection.

Strength of Grouted Connections

Other aspects of System 2 frames which have been of concern are the performance of grouted column bars and horizontal joints between the columns and the precast beams, and the performance of column ties around the grout-filled ducts which are oversized to provide construction tolerances (see Figures 3.6(c) and 3.9). Laboratory testing in New Zealand [3.14, 3.15, 3.16], summarised in Appendix B2, has indicated that

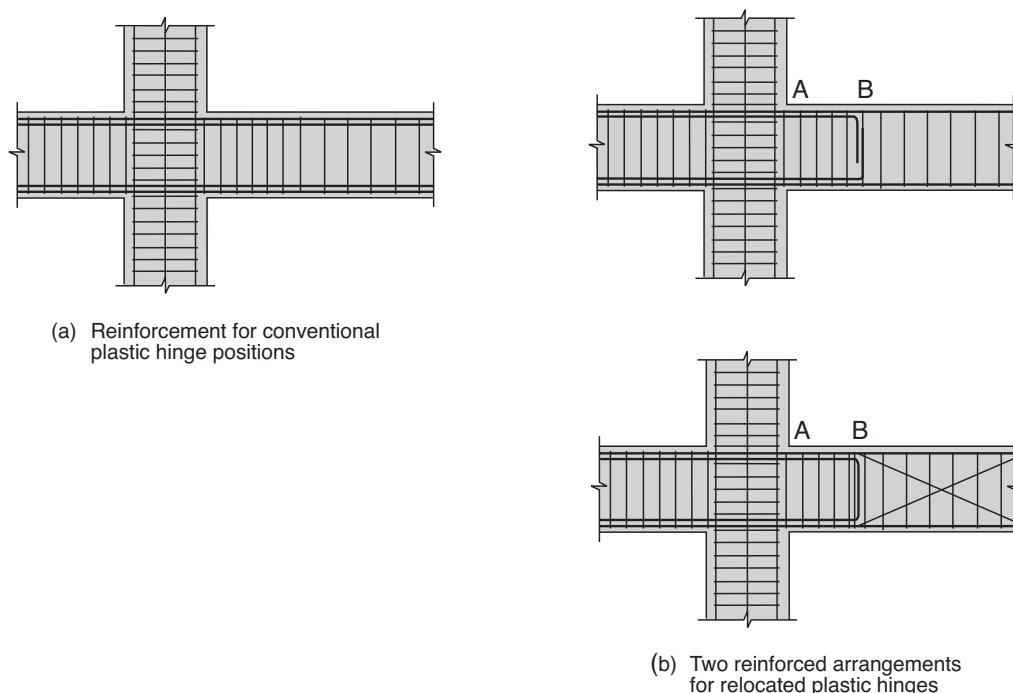
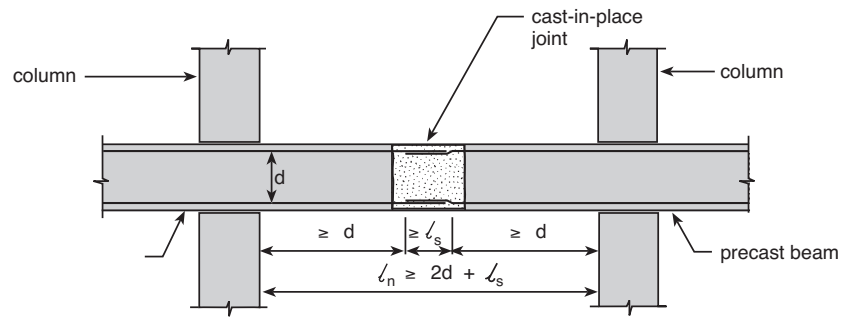
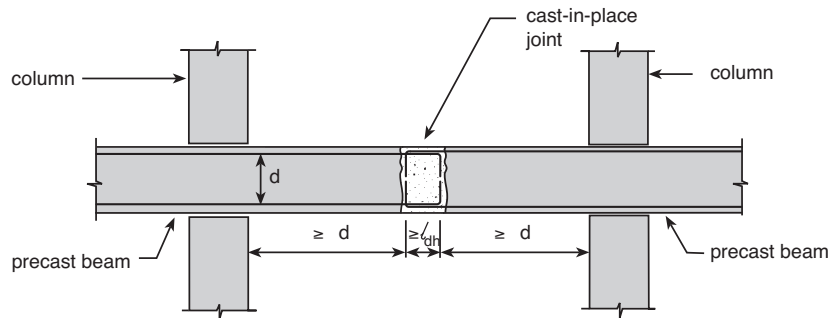


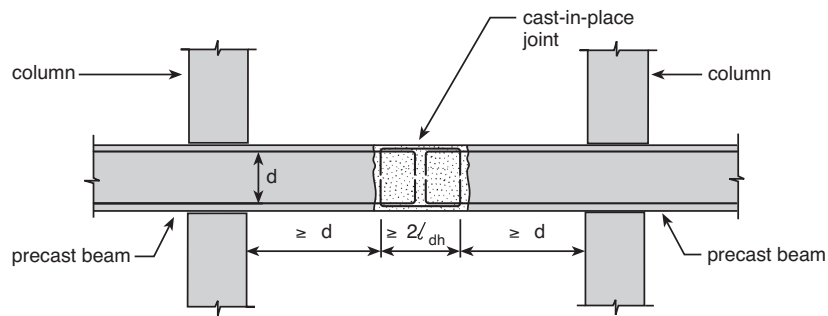
Figure 3.7: Relocated plastic hinge design for moment-resisting frames dominated by seismic loading



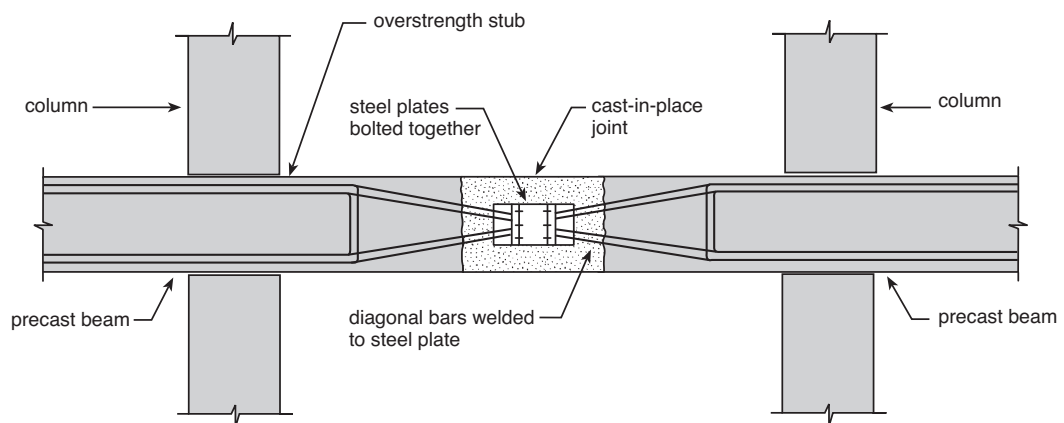
(a) Conventional Straight Bar Lap



(b) Hooked lap



(c) Double hooked lap



(d) Diagonal Beam Reinforcement

Note: Transverse reinforcement is not shown

Figure 3.8: Some details for midspan connections for beams which have been used in New Zealand

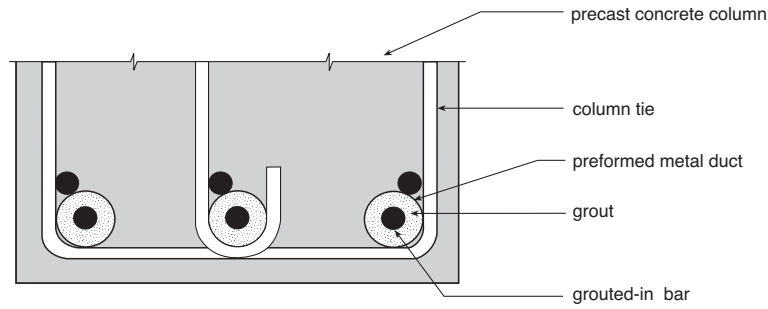


Figure 3.9: Part column section with grouted-in bars

provided design and construction are adequate, these aspects are satisfactory. The performance of this system was shown to be similar to that of a conventional cast-in-place joint.

3.4 System 3 - Precast T or Cruciform Shaped Units

A further possible structural arrangement incorporating T-shaped precast concrete elements is shown in Figure 3.1(c). The vertical column bars in the precast T units are connected using grouted steel sleeves. At the midspan of the beams, bottom bars can be spliced in a cast-in-place concrete joint or connected by welding to steel plates which are bolted together. An alternative to the T-shaped units is to use cruciform-shaped precast concrete units with joints between columns occurring at mid-height of the storeys. Precast floor systems can be used as with the other systems.

An advantage of System 3 is the extensive use of precast concrete possible, and the elimination of the fabrication of complex reinforcing details on the building site. A possible constraint is that the precast elements are heavy and crane capacity may be an important consideration.

Figure 3.10 shows the details of a precast concrete T-shaped unit used in the perimeter frame of a building in which plastic hinging was designed to occur in the diagonally reinforced beam regions away from column faces. Grouted steel sleeves were used to splice column bars at interfaces between units above the joint core, and high strength friction grip bolts were used to connect beam bars which were welded to steel channels at midspan. As for Figure 3.8(d), extreme care is required for the welded connection detail and the detail at the bends of the diagonal bars.

Figure 3.11 shows a perimeter frame of a building constructed using precast concrete cruciform-shaped units two storeys in height, with grouted steel sleeves

connecting column bars at mid-storey height and hooked splices connecting beam bars in cast-in-place concrete joints at midspan.

3.5 Low Frames with Strong Beam-Weak Column Design

Systems 1, 2 and 3 described in Sections 3.2, 3.3 and 3.4 have typically been used for strong column-weak beam design.

For one- or two-storey frames a strong beam-weak column design concept is permitted [3.11, 3.12]. Hence for such low frames the post-elastic deformations in a major earthquake can be designed to occur by plastic hinges forming at the column ends. For a strong beam-weak column design, System 1 of Figure 3.1(a) would be suitable. Also suitable would be System 3 but with cruciform-shaped units as in Figure 3.11, with connections between precast elements at mid-height of columns and at midspan of beams, rather than T-shaped units, in order to keep column splices out of potential plastic hinge regions.

A further possible arrangement of precast elements for such a design is shown in Figure 3.12. For this building, two-storey columns are precast in one length for the full height of the frame and the beams are precast to occupy about the middle 60% of each span. Top and bottom beam bars protruding from precast beam elements are spliced in end regions of beams in cast-in-place concrete. These bars are spliced with lengths of beam bar which are cast passing through the precast columns.

3.6 Mixed Precast Prestressed Concrete and Cast-in-place Reinforced Concrete Moment-resisting Frames

A further building system which has become popular

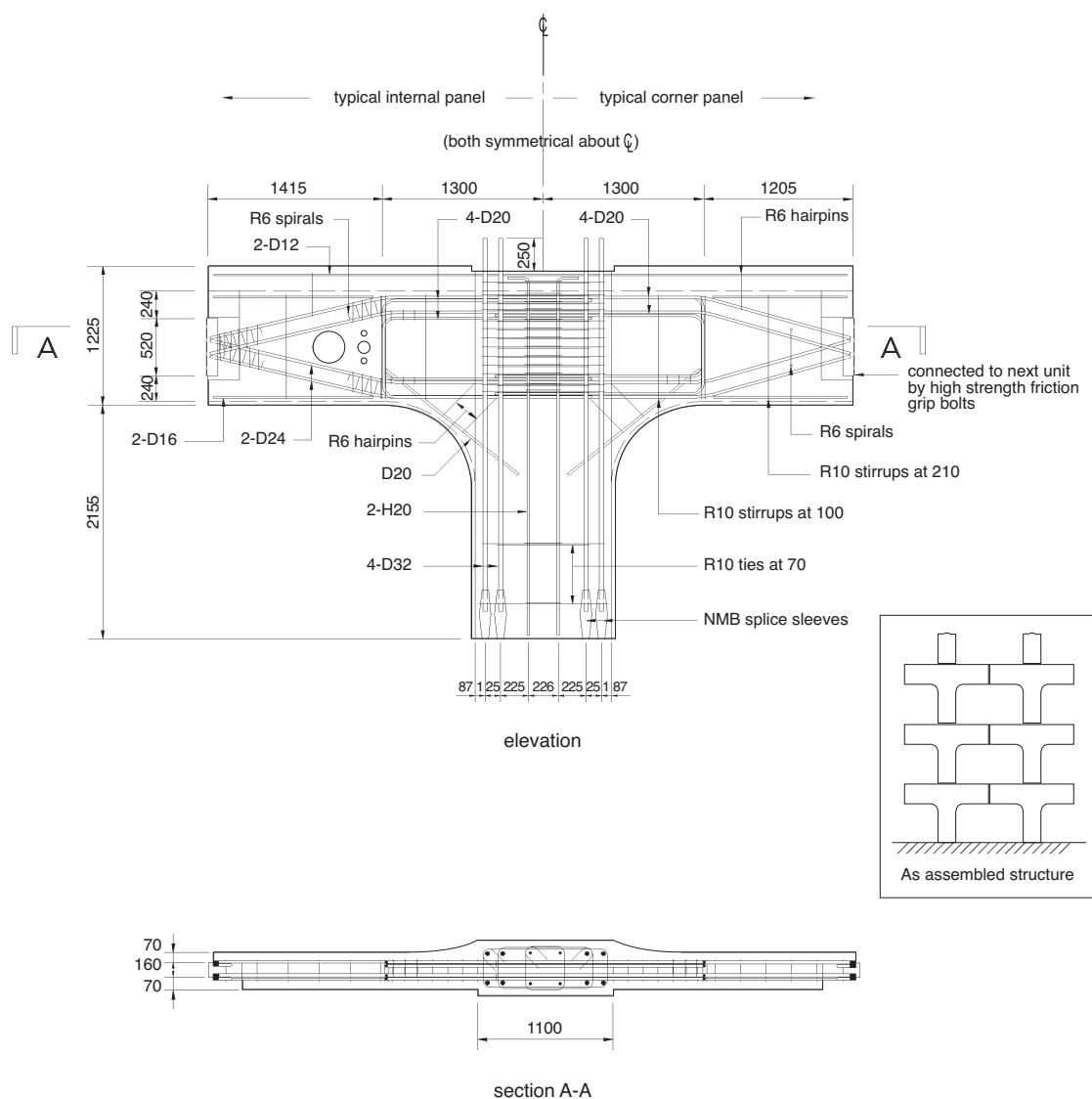


Figure 3.10: Details of precast concrete T-shaped units of the perimeter frame of a reinforced concrete building

in recent years involves the use of precast concrete beam shells as permanent formwork for beams. The precast beam shells are typically pretensioned prestressed concrete U-beams and are left permanently in position after the cast-in-place reinforced concrete core has been cast. The precast U-beams support the self weight and construction loads and act compositely with the reinforced concrete core when subjected to other loading in the completed structure.

Precast U-beams are generally not connected by steel to the cast-in-place concrete of the beam or column. Reliance is normally placed on the bond between the roughened inner surface of the precast U-beam and the cast-in-place concrete core to achieve composite action. Occasionally, protruding stirrups or ties from the U-beams have been used to improve the interface shear strength.

The typical structural organisation of a building floor and moment-resisting frame system incorporating precast pretensioned U-beam units is shown in Figure 3.13. Figure 3.14 shows a ductile frame under construction using precast U-beams.

This form of composite beam construction has been used in multi-storey moment-resisting framed structures. In this application, the composite beams will be required to develop ductile plastic hinges during major earthquakes.

Doubts have been expressed by some designers and checking authorities concerning the ability of this form of composite construction to be able to perform as ductile moment-resisting frames. It had been felt that cracking may concentrate in the beam at the column



Figure 3.11: Perimeter frame of a reinforced concrete building constructed using two-storey high precast concrete cruciform units, Unisys House, Wellington

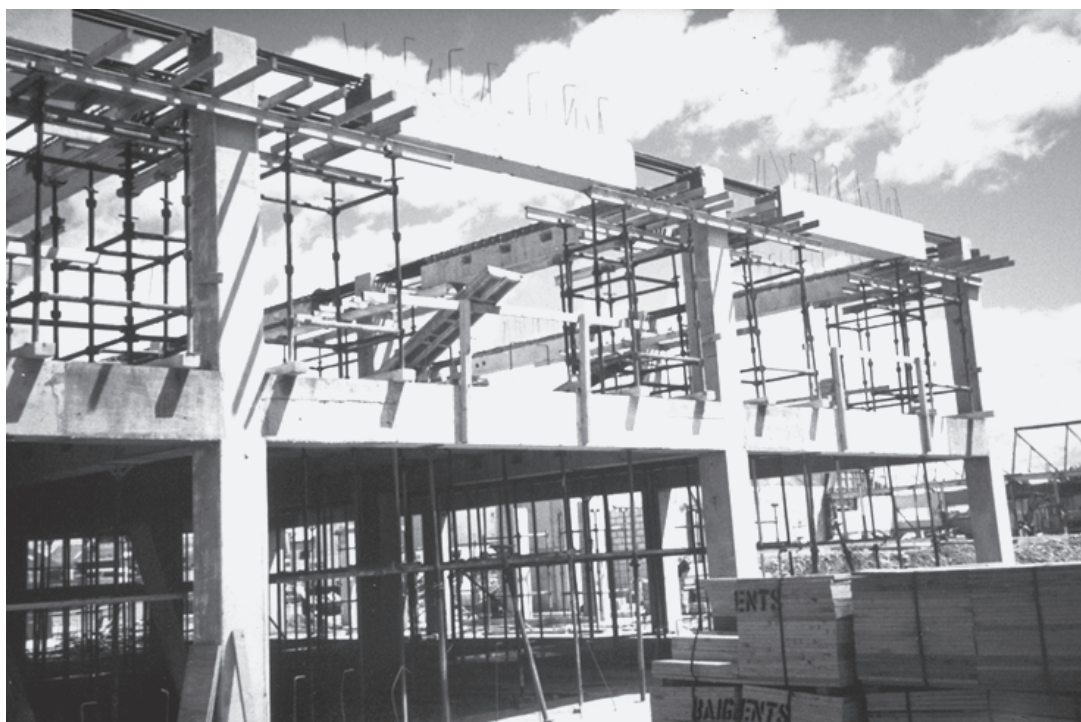


Figure 3.12: Two-storey frame with columns precast in one length for the full height and cast-in-place spliced joints at the ends of precast beams

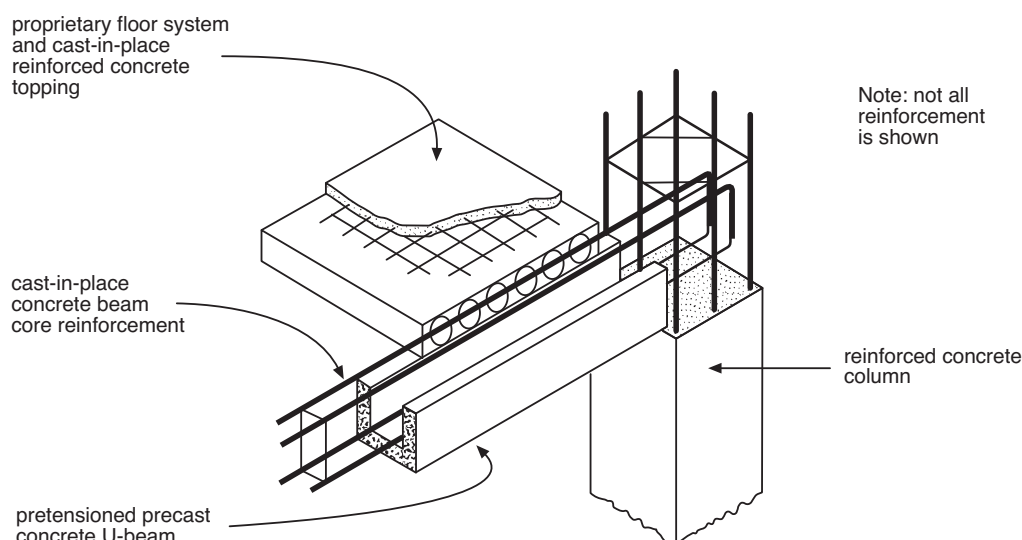


Figure 3.13: Construction details of a structural system involving precast concrete beam shells and cast-in-place reinforced concrete

face at the discontinuity caused by the end of the precast U-beam. However, tests have demonstrated [3.17] that during severe seismic loading there is a tendency for the plastic hinging to spread along the cast-in-place reinforced concrete core within the precast U-beam due to breakdown of bond. Hence plastic hinge rotation does not concentrate in the beam at the column face and no undesirable concentration of curvature results. Seismic design recommendations for such construction are available [3.17]. It is considered that this type of construction is suitable for ductile moment-resisting frames.

It is very important to ensure during construction that the inside surface of the shell beams is clean when the cast-in-place concrete is placed, otherwise sufficient bond between the shell and core cannot develop. A site failure of a beam due to lack of bond because of a dirty interface has been observed.

3.7 Pinned Joints

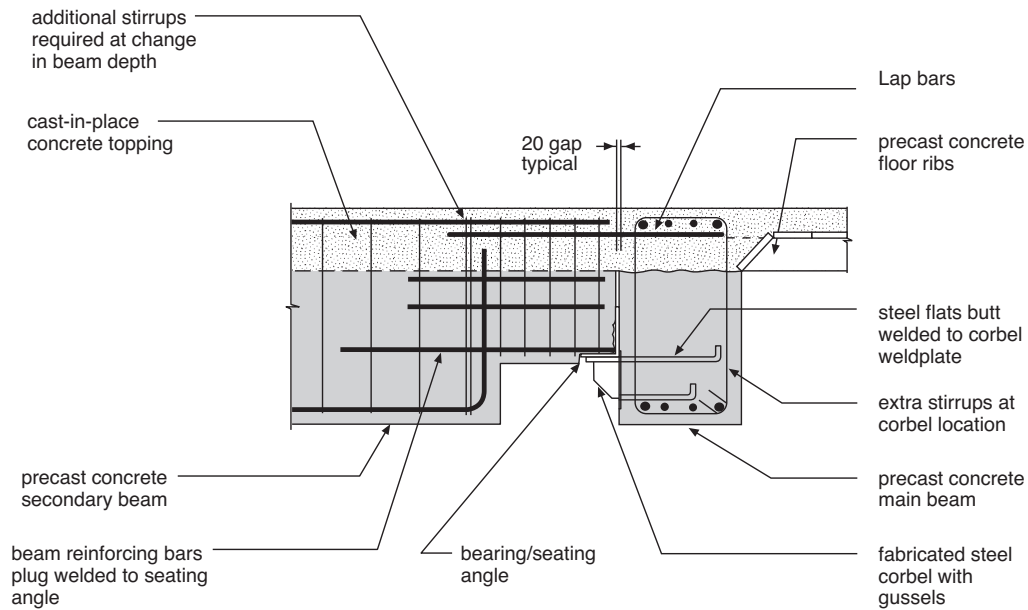
Pinned joints can be used to connect secondary beams to primary beams and sometimes at beam-column joints to reduce the moment input from the beam to the column. Examples of pinned joints at secondary beam to main beam connections are shown in Figure 3.15. The typical 20 mm tolerance gap, between the end of the secondary beam and the side of the main beam, implies that the precaster and contractor may be required to work to more stringent tolerances than those specified in NZS 3109 [3.18]. Also, the welding of reinforcing bars to RHS, or bars to the seating angle, is

critical. Weld throat thicknesses need to be carefully monitored.

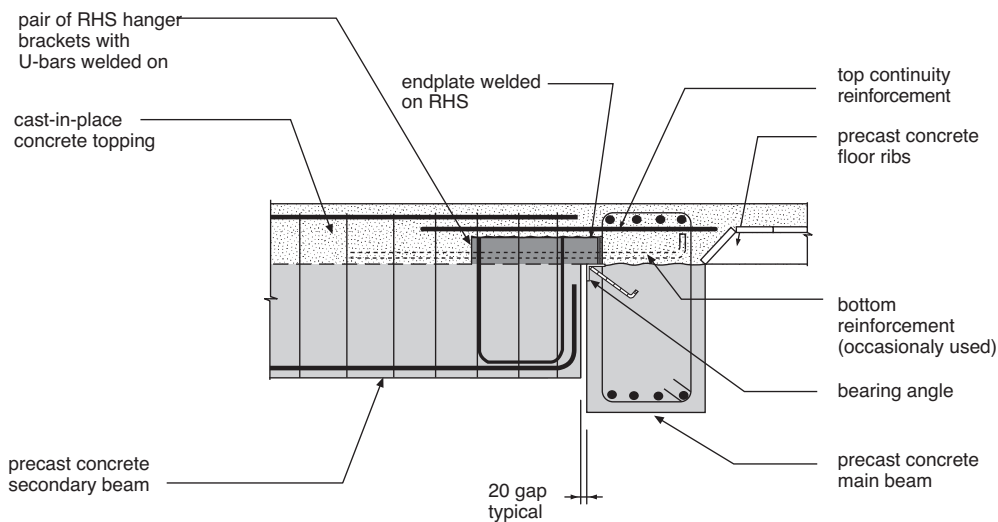
In many instances beams that are supported each end



Figure 3.14: Construction of a ductile frame using precast concrete shell beams



(a) Using Steel Corbel Angle Bracket



(b) Using RHS Hanger Brackets

Figure 3.15: Example of secondary beam to main beam connections

on pairs of RHS hangers or on wide corbel angle brackets will only bear on the two diagonally opposite RHS hangers or on one side of the wide corbels. This is because the various component parts are not cast or fabricated perfectly square and true. The bearing surfaces should be shimmed as necessary to ensure even bearing on all surfaces. However, designers should take into account in the design of the components this common miss-fit scenario.

Bottom reinforcement by tying the secondary beam to the main beam is occasionally used to improve the overall robustness of the “Pin Joint” details.

3.8 Composite Reinforced Concrete Moment-resisting Frames of Limited Ductility

Cast-in-place ductile reinforced concrete structural walls in a building can be designed to resist almost all of the seismic loading acting on the building, if they are very stiff compared with the frames in the building. Then the frames in the building are present mainly to carry the gravity loading. Such moment-resisting frames can be designed for limited ductility, using less transverse reinforcement than for ductile frames, providing

it can be shown that when the walls have deformed inelastically to the required displacement ductility factor during severe loading, the ductility demand on the frames is not large. This is possible when the frames are much more flexible than the walls.

A building so designed is illustrated in Figure 3.16. The central cast-in-place reinforced concrete walls forming the service core of the building were designed to resist the seismic loading. The perimeter frame of precast concrete beams, and shell columns infilled with cast-in-place concrete, was designed mainly for gravity loading.

3.9 Industrial Buildings

Totally precast concrete frames have proved to be suitable for industrial buildings with a large degree of repetition, or enclosing processes giving off corrosive vapour (for example, pulp and paper processing). Typically the frame would consist of precast pretensioned, prestressed concrete members. Normally the roof beam is simply supported at its ends on cantilever columns, and the seismic loading is resisted by the cantilever columns.

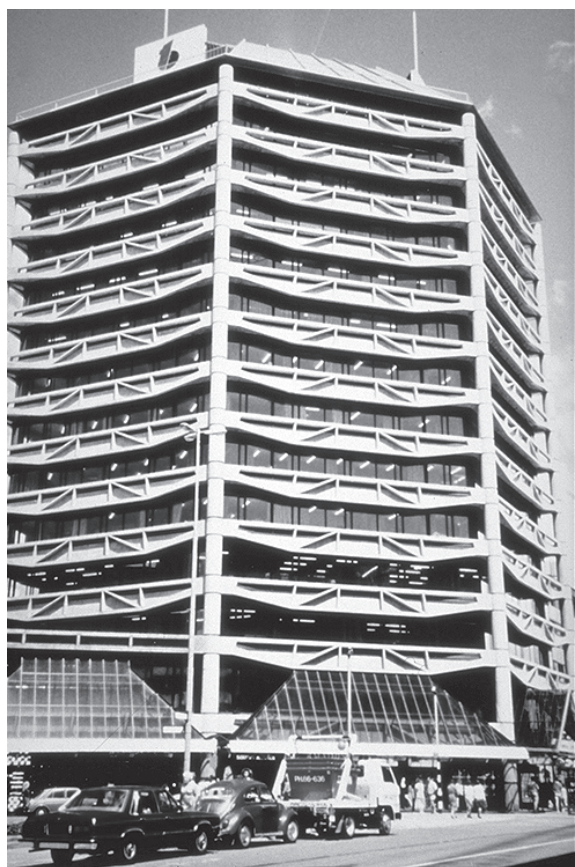


Figure 3.16: Precast concrete perimeter frame in building with seismic loading resisted mainly by interior core of cast-in-place walls

Figure 3.17 shows photographs of some examples of single-storey industrial buildings which have been constructed in New Zealand.

3.10 Frame Connections in North American and Japanese Practice

The New Zealand design approach for moment-resisting frames has been to use cast-in-place concrete connections between precast concrete elements and to seek structural behaviour as if of monolithic construction.

Moment-resisting frames incorporating precast concrete elements have had very little use in the United States. United States practice has been mainly to use “dry connections” formed by welding, dry packing and grouting. Dry connections do not always behave as if part of monolithic construction. The Prestressed Concrete Institute of the United States has sponsored research on moment-resisting connections. The emphasis of the early research work has been on dry connections. A summary of some of this work is presented in References 3.19 and 3.20. In most cases where welding or bolting was used, there was a lack of ductility. It is considered that as long as welding and bolting is used in the critical region it will be extremely difficult to ensure ductile behaviour in primary seismic load-resisting elements.

In Japan, structural steel or steel-reinforced concrete has traditionally been used for tall building structures. However, in the 1980s the technical feasibility and economics of reinforced concrete for tall buildings was realised and now reinforced concrete moment-resisting frames typically 20- to 30-storeys in height are commonly-used in Japan for apartment buildings. Many incorporate precast concrete elements. The types of moment-resisting frame connections between precast members used in Japan are similar to those in use in New Zealand. Examples of Japanese practice are given by Kurose et al [3.21] and Kanoh [3.22]. It is noted that a considerable amount of laboratory testing is being conducted in Japan by construction companies.

3.11 Recommendations

3.11.1 Arrangements of Precast Concrete Members in Moment-resisting Frames

Several possible arrangements of precast concrete members and cast-in-place concrete forming ductile moment-resisting frames have been used in New Zealand (see Sections 3.2 to 3.6). The design aim has been to



Figure 3.17(a): Hamilton showgrounds building (bolts at tops of cantilever columns pass through Y-shaped rafters - built in the late 1960s)



Figure 3.17(b): Aluminium smelter, Tiwai point (cantilever columns and pinned rafters built in the early 1970s)



Figure 3.17(c): Caxton paper mills, Kawerau (built in mid-1980s)

connect the precast concrete elements together so that the seismic performance will have the essential integrity of a monolithic cast-in-place concrete structure.

3.11.2 Frames Formed from Precast Beams Spanning between Columns with Cast-in-place Beam-column Joints

The column dimensions need to be large enough to accommodate the required development lengths of the beam longitudinal reinforcement and to avoid congestion of the hooked bottom bars in the beam-column joints as far as possible. The ends of the precast beams should be roughened or keyed to assist shear transfer across the vertical cold joints at the column faces (see Section 3.2).

3.11.3 Frames Formed from Precast Beams Extending from Midspan to Midspan with the Longitudinal Column Bars Grouted in the Beam-Column Joints

The splicing of the longitudinal beam bars in a cast-in-place joint in the midspan regions can be achieved by either straight or hooked laps, or by diagonal reinforcement welded to plates, which are bolted together. Splices in longitudinal beam bars should not commence within d of the column face, where d is the effective depth of the beam. The grouting of horizontal joints between the columns and the soffits of the precast beams, and the grouting of the longitudinal column bars passing through precast beams, when

carried out satisfactorily, will result in performance of the joint similar to that of a cast-in-place concrete joint. However, strict grouting procedures need to be followed (see Section 3.3).

3.11.4 Frames Formed from T-Shaped and Cruciform Shaped Precast Units

Adequate splicing of the beams of precast units in the mid-span regions, and the columns at the beam-column joints or at the mid-height regions of storeys, can be achieved (see Section 3.4).

3.11.5 Frames Formed from Mixed Precast Prestressed and Cast-in-Place Concrete

Moment-resisting frames constructed from precast concrete beam shells and cast-in-place concrete can be designed to perform satisfactorily. Special attention in design needs to be paid to the performance of the precast concrete shell and its cast-in-place concrete core (see Section 3.6).

3.11.6 Low Frames With Strong Beam-Weak Column Design

The above structural arrangements have been used mainly for strong column-weak beam designs. For one- or two-storey frames where a strong beam-weak column design is permitted, only the arrangement with grouted column bars may be unsatisfactory (see Section 3.5).

3.11.7 Industrial Buildings

Precast concrete has been successfully incorporated in frames used for industrial buildings in seismic areas.

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Chapter 4

Structural Wall Elements

4.1 Introduction

The usefulness of structural concrete walls in buildings has long been recognised in New Zealand as they have been found to be efficient in resisting lateral forces due to wind and earthquakes. Their large inherent stiffness means that drift is limited during a severe earthquake, thus providing a high degree of protection against non-structural damage. Comprehensive seismic design provisions for cast-in-place reinforced concrete walls are given in the New Zealand Concrete Structures Standard [4.1].

Tests [4.2] have shown that well-detailed walls give the required strength and ductility. It is recognised that well-proportioned, ductile, cast-in-place reinforced concrete, coupled cantilever walls, probably form the best earthquake resistant structural system available in reinforced concrete [4.3]. In these, the coupling beams can be designed so that most of the hysteretic energy dissipation occurs in them, thereby limiting the damage that is sustained by the vertical load-resisting wall elements.

Although most seismic resistant structural concrete walls for multi-storey buildings in New Zealand are of cast-in-place construction, there has been some construction utilizing precast concrete wall panels. This chapter examines and makes recommendations on the use of precast concrete in structural wall elements.

4.2 Types of Precast Concrete Structural Wall Construction

Precast concrete structural wall construction usually falls into two broad categories [4.4]:

- “monolithic” wall systems; or
- “jointed” wall systems.

The distinction between these two types of construction is based on the design of the connections between individual precast concrete panels, which when jointed together form the structural wall. For “monolithic” wall systems, connections are designed as “strong” connections, so their elastic limit is not exceeded in satisfying the building’s ductility demands. Alternatively, in the case of “jointed” wall systems, connec-

tions may be designed as “ductile”, with energy dissipation occurring in the connection, thereby contributing to the building’s overall ductility.

It is possible to use a combination of monolithic and jointed details. For example, multi-storey residential construction in Japan, America and Yugoslavia often utilises “monolithic” vertical joints and “jointed” horizontal joints [4.5].

Some examples of monolithic and jointed wall connections used in Yugoslavia, Japan and USA are shown in Figures 4.1 to 4.8.

The design of the connections between precast concrete wall elements largely depends on the type of construction. However, factors such as capacity design principles, cartage, craneage capacity, erection procedures, volume changes from creep, shrinkage and temperature, need to be taken into consideration.

A common precast concrete wall construction method for single and two-storey buildings in New Zealand uses the tilt-up wall construction technique. With tilt-up construction, relatively large wall panels are cast horizontally on top of concrete floor slabs or casting beds adjacent to final wall panel positions. When the concrete used has gained sufficient strength for the wall panels to remain uncracked during lifting operations, the walls are tilted up and lifted into their permanent positions. Generally tilt-up walls are secured to the adjacent structural elements with jointed connections comprising various combinations of concrete inserts, bolts, weldplates, angle brackets and lapped reinforcement splices within cast-in-place joining strips. Reference 4.6 provides full details for tilt-up wall design and construction for New Zealand conditions. Reference 4.21 discusses recent research into the seismic performance of a wide range of panel-to-panel connections and floor-to-panel connections.

Reference 4.22, a report by the PCI Ad Hoc Committee on Precast Walls, discusses recent experience, research, and design regulations employed in North America for the use of structural (shear) precast concrete walls. Included in this comprehensive report, are references to a number of papers describing the performance of precast walls during major seismic events over the last 25 years. This report and references within

will be of considerable interest to designers dealing with the seismic design of structural wall elements.

4.3 Monolithic Wall Systems

4.3.1 General

In monolithic construction, precast elements are joined

by reinforced concrete connections possessing stiffness, strength, and ductility believed to be comparable to cast-in-place concrete. Monolithic precast wall systems are often used as a cost-effective alternative to cast-in-place ductile cantilever shear walls or ductile coupled shear walls. Designers expect these to be capable of sustaining the significant inelastic deformations required of a ductile structure.

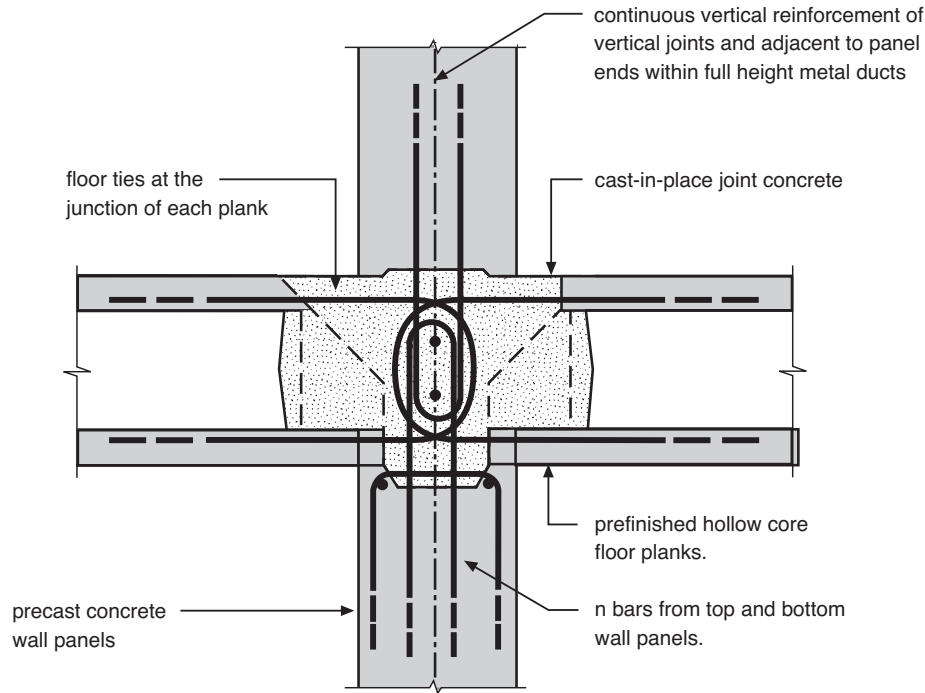


Figure 4.1: Nearly monolithic precast wall construction horizontal joint of the SCT System, Yugoslavia (Reference 4.12)

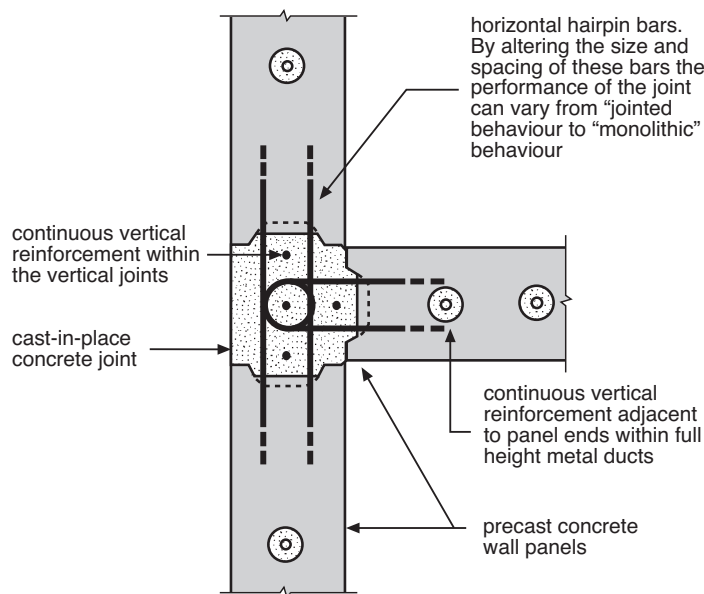


Figure 4.2: Nearly monolithic precast wall construction vertical joint of the SCT System, Yugoslavia (Reference 4.12)

The satisfactory performance of well-detailed cast-in-place concrete walls has been established by exhaustive laboratory testing and observation of their behaviour during earthquakes. If designers of precast concrete wall systems detail the connections between the components to possess stiffness, strength and ductility comparable to cast-in-place concrete, then the completed walls can be expected to perform satisfactorily.

Care is needed, however, in design and construction as indicated below.

4.3.2 Design of Monolithic Walls

In the absence of a New Zealand code of practice written specifically for precast concrete construction, designers usually design monolithic precast structural walls to the requirements of NZS 3101 [4.1].

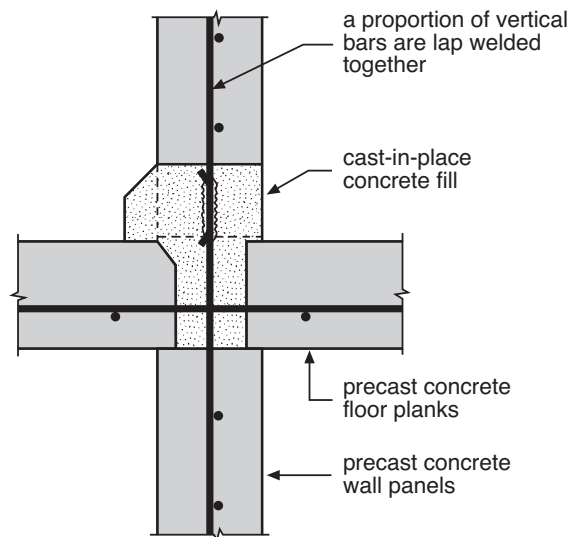


Figure 4.3: Jointed precast wall construction example of horizontal joint for 2-storey houses in Japan (Reference 4.9)

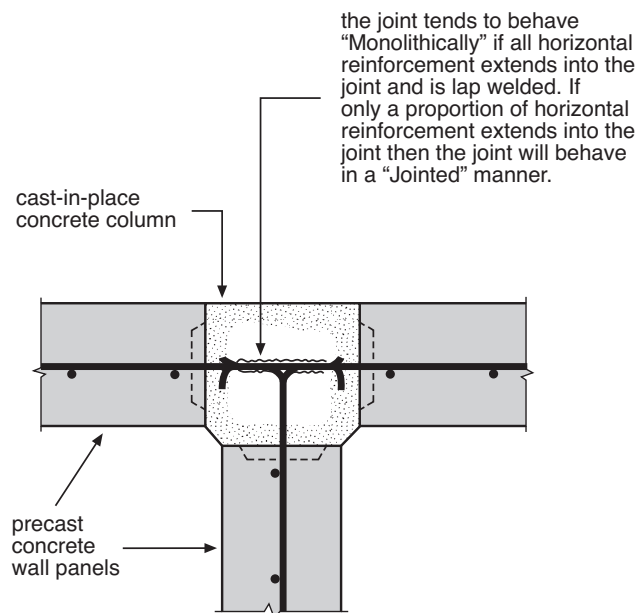


Figure 4.4: Jointed or monolithic precast wall construction example of vertical joint for 2-storey houses in Japan (Reference 4.9)

Since NZS 3101 has been developed principally for cast-in-place concrete construction, there are some difficulties in applying this standard to precast concrete wall designs. Care is needed when applying the dimensional limitation rules. In NZS 3101 it is assumed that structural walls have full flexural continuity about their minor axis at each level of lateral restraint (floor level). With precast concrete walls there may be a major discontinuity in the flexural

strength about their minor axis at each horizontal joint location. Designers are therefore encouraged to adopt a conservative approach to the ratio of the clear storey height to the wall thickness.

NZS 3101 requires two layers of longitudinal (vertical) reinforcement when structural walls are more than 200 mm thick. This can be easily achieved over the height of the wall panels, but having two layers of reinforce-

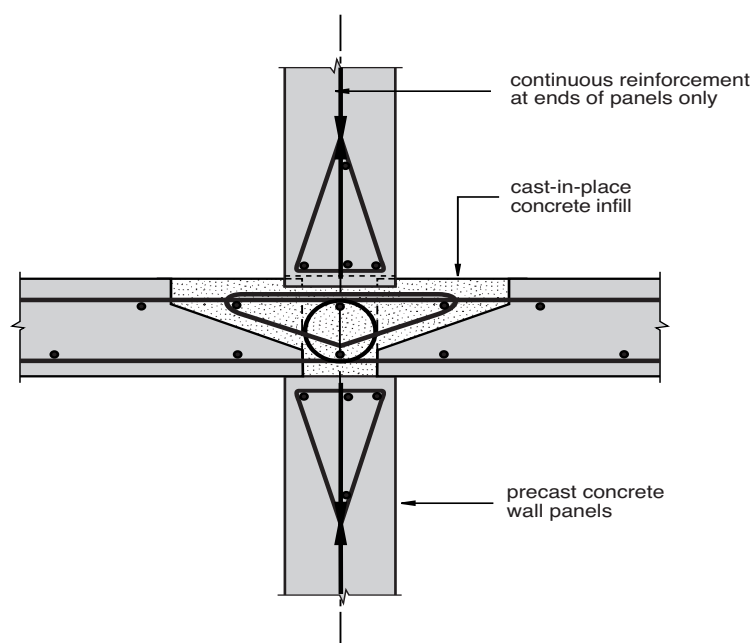


Figure 4.5: Jointed precast wall construction horizontal joint of the “Rad” System, Yugoslavia (Reference 4.15)

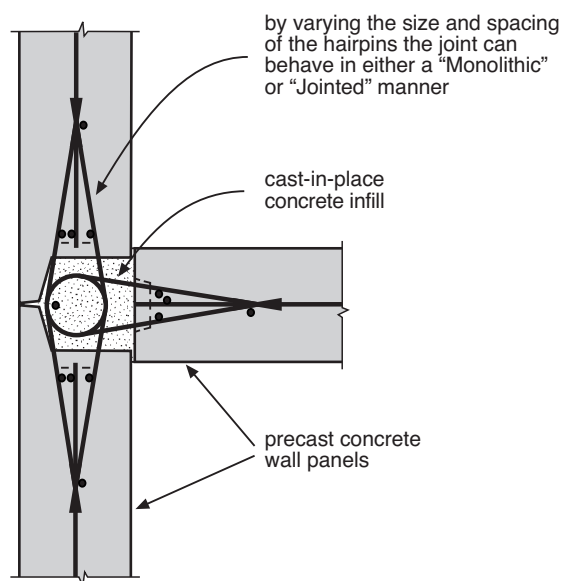


Figure 4.6: Jointed or monolithic wall construction vertical joint of the “Rad” System, Yugoslavia (Reference 4.15)

ment at horizontal panel joints is usually impractical. Furthermore, NZS 3101 has a restriction that the diameter of bars used in any part of a wall shall not exceed one-tenth of the thickness of that wall. This limitation can often be a major constraint when using central vertical lap bars at horizontal joints that lap with pairs of smaller bars placed either side of the central bar (see Figure 4.10, Section A-A).

NZS 3101 typically requires staggering of the splices of the principal vertical flexural tension reinforcement within potential plastic hinge zones. This is very difficult to achieve with practical details. Usually, laps in the cast-in-place concrete bandage joints and the cast-in-place end wall thickenings are staggered in level from those in the precast wall sections. Yielding of the bars splicing the precast wall sections together is

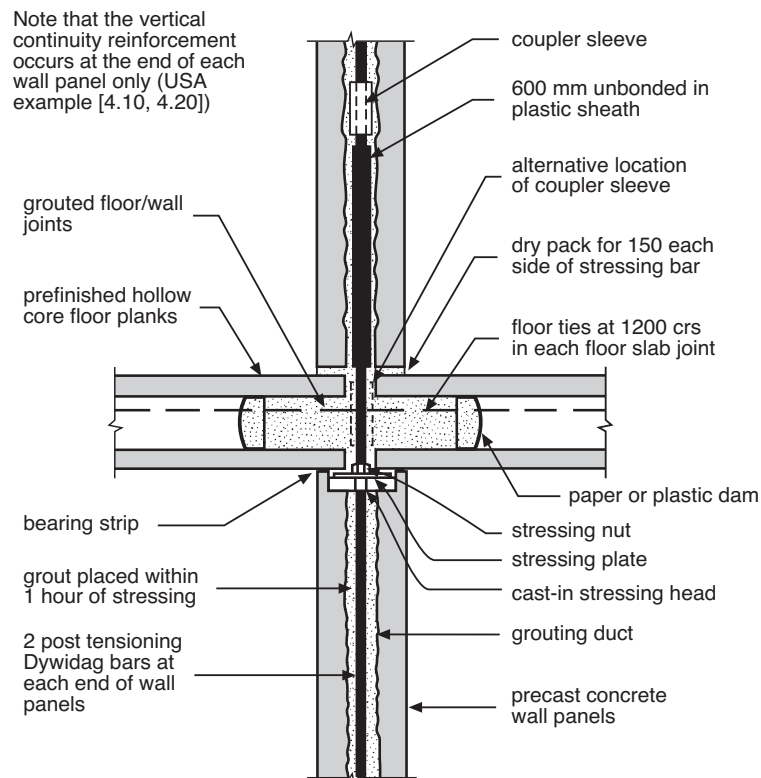


Figure 4.7: Jointed precast wall construction horizontal joint — platform type construction

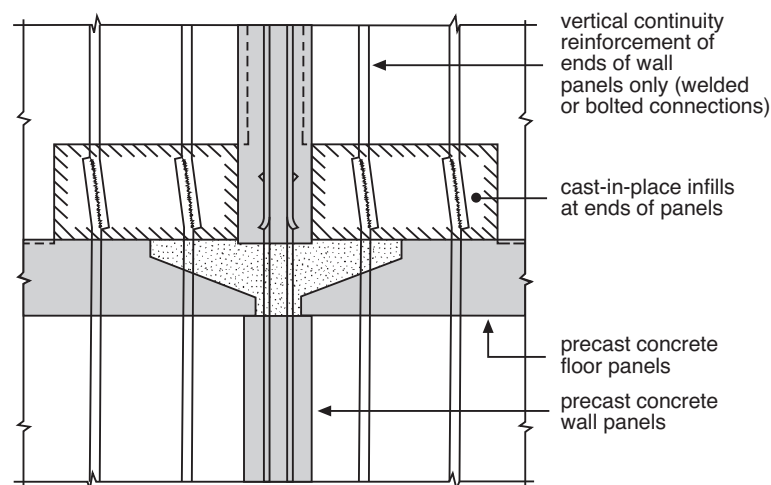


Figure 4.8: Jointed precast wall construction elevation of horizontal joint (Reference 4.8)

usually avoided by selecting a lap bar size and/or steel grade that is slightly stronger than the vertical bars that lap with it, i.e. one D24 (452 mm²) could lap with two D16 bars (402 mm²). This helps to minimise flexural cracking over the lapping region.

Research is therefore needed to check the sensitivity of monolithic precast concrete wall design and construction to the difficulties which have been identified.

4.3.3 Vertical Joints

Vertical joints between wall panels are typically cast-in-place “bandage” type joints. Horizontal reinforcing from a precast panel projects into the joint zone and is lapped with horizontal reinforcing from an adjacent panel. The amount and spacing of the horizontal shear reinforcement is established using capacity design principles and concrete code requirements [4.1]. The widths of the cast-in-place vertical joints are determined by concrete code requirements for lap lengths of the horizontal reinforcement. Typical details of monolithic vertical joints are shown in Figure 4.9.

A wide range of details are in common use in New Zealand. The design of the lap or splice regions should comply with the requirements of NZS 3101, or alternatively be laboratory tested. In many instances, the ends of monolithic precast walls have cast-in-place columns which improve the wall stability where flexural reinforcement and transverse stirrups and ties are concentrated.

4.3.4 Horizontal Joints

The vertical reinforcement in precast walls is usually lapped at horizontal joints. Proprietary grouted steel sleeve splices may be used for this purpose, or alternatively the lap can be formed by grouting a bar extending from one unit into a metal duct in the matching unit. The spacing of steel sleeve splices or metal ducts is usually at no more than 450 mm centres to comply with the maximum spacing provisions of NZS 3101. Some typical details of horizontal joints in monolithic construction are shown in Figure 4.10. Some steel sleeve splices have been cyclic load tested and comply with the concrete code requirements as high strength mechanical connectors suitable for use in a plastic hinge zone, provided that the grout used complies with the sleeve manufacturer’s specification [4.7].

When corrugated metal preformed pipe ducts are used, starter bars that project into the ducts are usually designed for a full lap length as defined in NZS 3101. Generally central starter bars are lapped with pairs of smaller bars, one on each face of the precast concrete wall section or, alternatively, all of the main flexural reinforcement is lapped on the precast concrete wall

centreline and some additional basketing cover reinforcement is provided.

The horizontal joint between precast concrete panels is usually scabbled or retarded and cement paste removed to provide appropriate surface roughness to avoid a sliding shear failure. The joint is then grouted using the method outlined in Chapter 6. When using steel sleeve splices, these are usually grouted individually. Foam plastic rings are used to seal the base of the steel sleeve splices to allow the separate grouting of the sleeves and the horizontal joint between the precast concrete panels.

4.4 Jointed Wall Systems

4.4.1 General

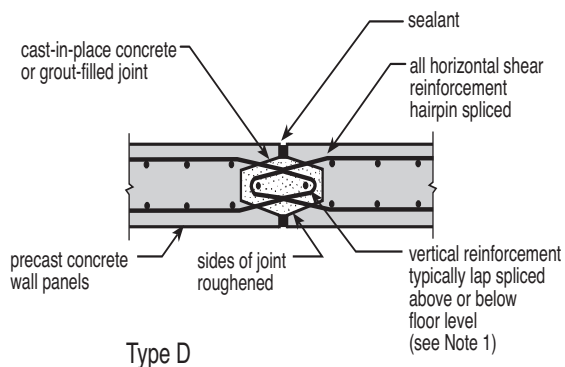
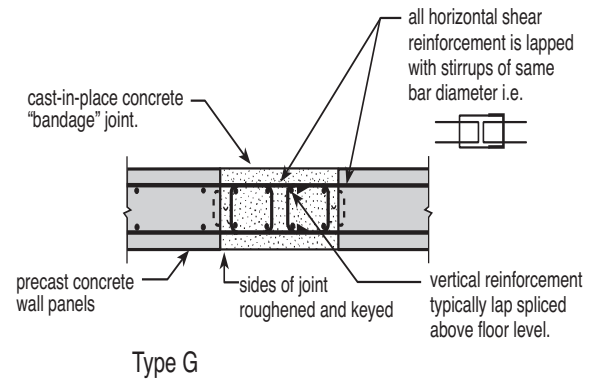
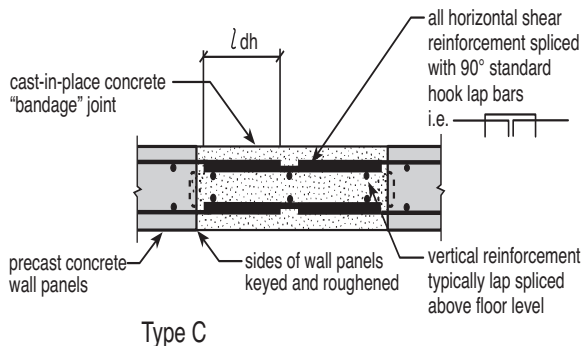
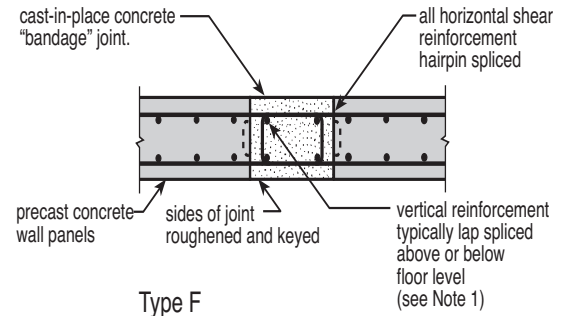
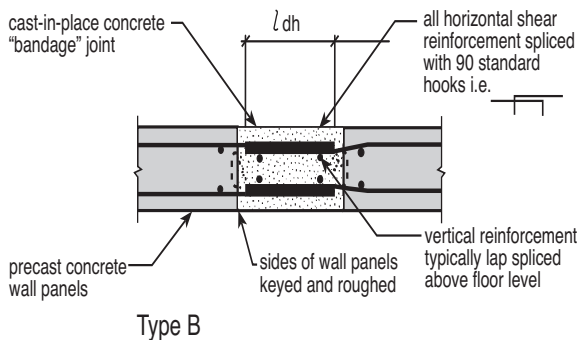
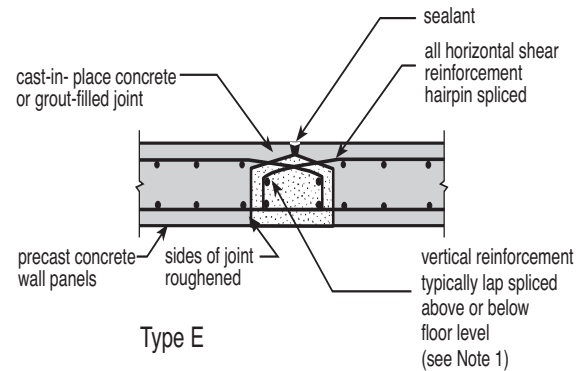
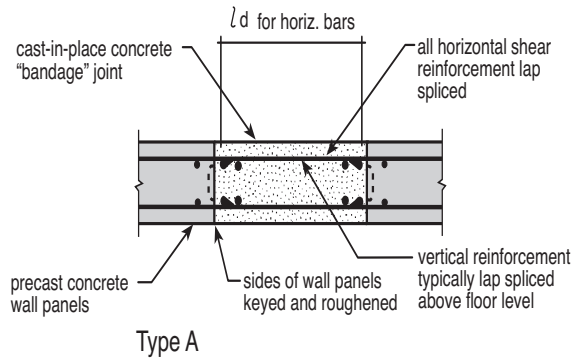
Jointed construction describes the connection of precast components, which result in planes of significantly reduced stiffness, strength, and ductility at the interface between adjacent precast concrete members. This type of precast concrete wall construction is not common for high rise construction in New Zealand but, because of potential economy in construction, requires consideration.

Design and construction of buildings up to eight storeys high utilising jointed precast wall systems is commonplace in countries such as Japan, America, Yugoslavia, USSR, Romania and Bulgaria [4.5]. Typically, this form of construction has been used for high density residential developments. Various forms of jointed precast wall systems are occasionally used in New Zealand for medium height commercial or industrial structures.

4.4.2 Analysis

Analysis assumptions commonly used for monolithic or cast-in-place wall structures are usually not suitable for jointed precast concrete wall systems. For the ultimate limit state, in which member strengths are assigned according to the relative elastic stiffness of the undamaged structure, there is a reliance on inelastic load redistribution to adjust for differences between actual and computed load paths. Jointed precast concrete structures that rely on discrete connectors of somewhat limited ductility have a relatively low degree of redundancy. Consequently, their load redistributing capacity may be insufficient to accommodate inaccuracies of the simplified analyses usually performed for monolithic wall structures.

Changes in the relative wall stiffness of jointed wall systems that occur as joints open and close during a severe earthquake response are often neglected. The



Note 1. Vertical joints shown as Types D, E and F need to be detailed with extreme care. Once the lapping bars have been overlapped the ability for lowering the wall panels over starter bars is very restricted. These details will typically work only when grouted steel splice sleeves are used to splice the vertical flexural reinforcement and when the laps of the vertical bars in the "bandage" joints are made below floor level.

Joint D is not preferred because joint reinforcement and concrete infill cannot be inspected.

Figure 4.9 : Monolithic precast concrete wall construction vertical joints

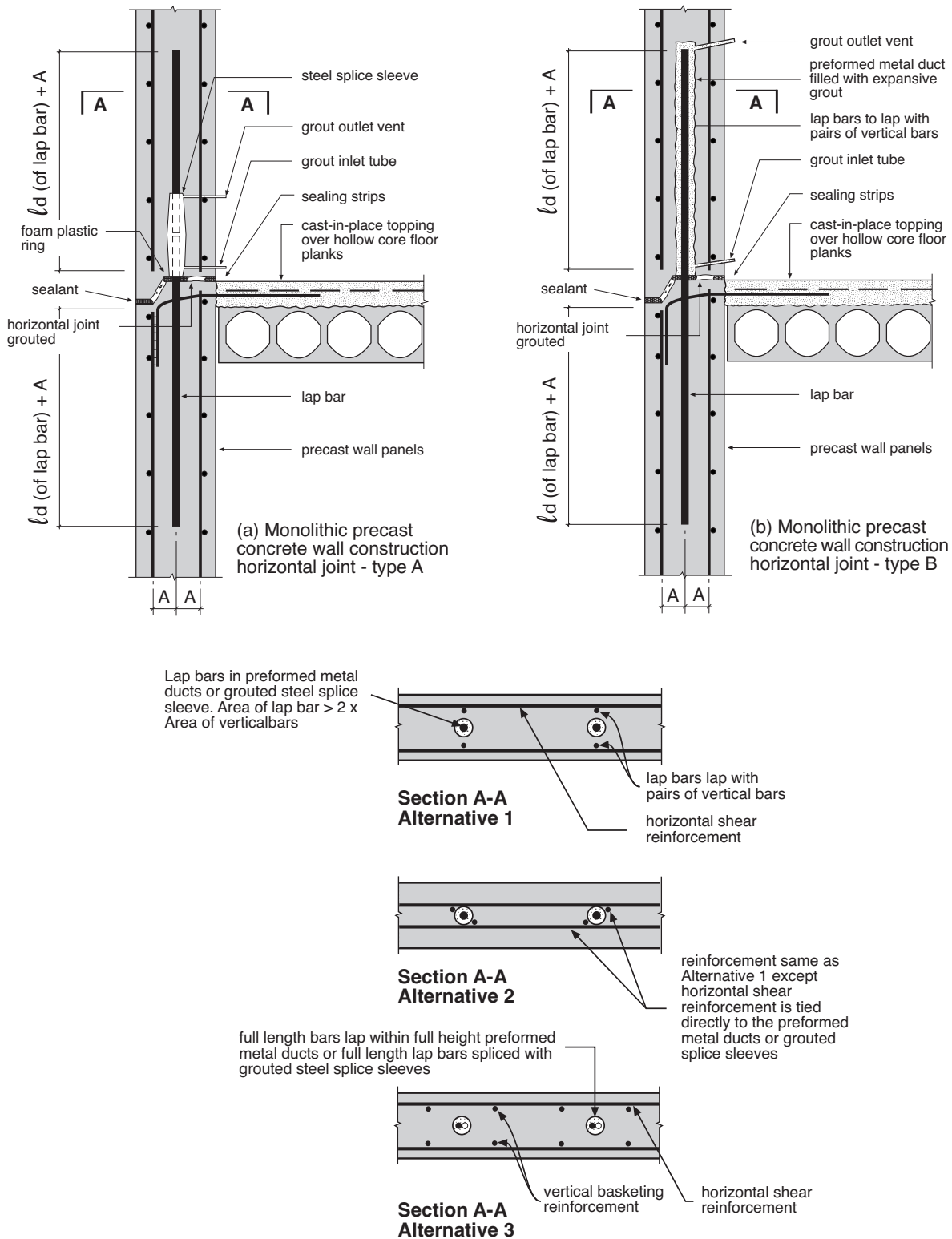


Figure 4.10: Monolithic precast concrete wall construction

assumption that foundations and floor diaphragms are rigid may cause further inaccuracies [4.4]. Designers should be cautious in their approach to the design of jointed precast concrete panel systems. Inelastic actions of the joints should be considered and a rational approach to the design of the connections formulated.

4.4.3 Design and Detailing

There has been considerable research and prototype testing of jointed panel systems overseas and a wide range of jointing details have been developed. Many papers have been written and readers are directed to references such as 4.8 to 4.18.

Some of the usual features of jointed panel systems, sometimes referred to as “large panel building systems”, are:

- they are normally associated with medium rise residential construction;
- the floor spans are typically less than 6 m;
- the floors are either of precast concrete slabs with cast-in-place toppings or large (room size) prefinished precast concrete floor slabs;
- the vertical joints often incorporate insitu “band-ages” with non-standard laps of nominal amounts of horizontal reinforcement;
- the horizontal joints usually have only two discrete welded connections per panel, above or below floor level;
- the horizontal joints are usually dry packed for most of their length;
- the panels are often rebated at the base in the vicinity of the connections to allow for the construction of cast-in-place concrete shear keys;
- the design shear stresses and flexural demands on panels are usually low (mainly because there is usually a large number of shear walls of adequate length resisting lateral loads in both principal directions); and
- the inelastic demand on the shear walls is usually accommodated by panel rocking and/or sliding shear mechanisms.

4.5 New Zealand Examples

4.5.1 Police Training College Residential Buildings

These three-storey buildings have external and close centred internal walls forming a jointed wall system of precast concrete. The walls are of “I” or channel cross section with each being made up of three precast panels (two flanges and one web) per storey height (Figure 4.11). The wall sections were placed one above the other up the height of the building, separated by cast-in-place concrete floor slabs, and connected together so as to form vertical cantilevers to resist seismic loads.

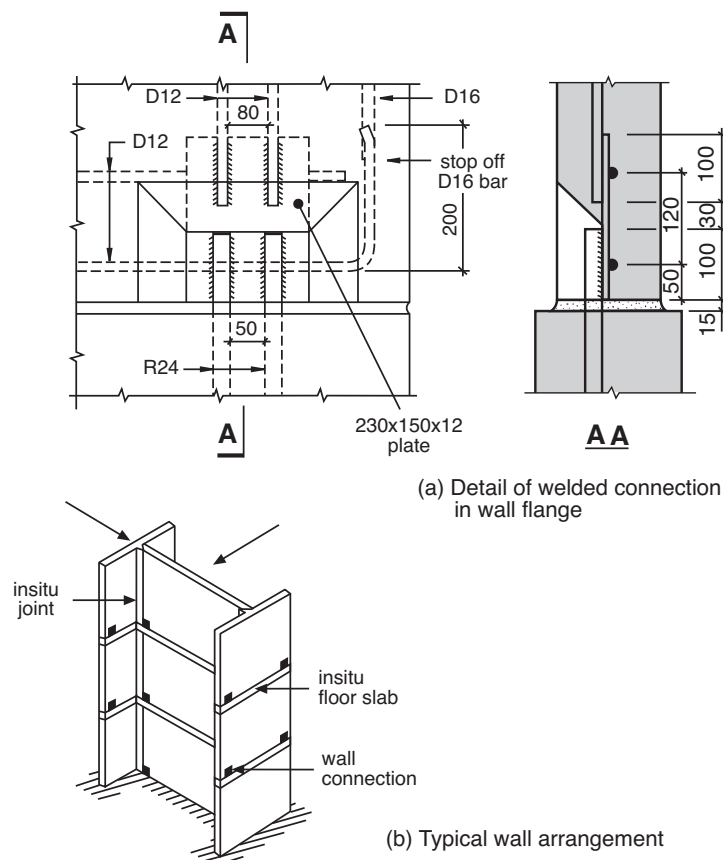


Figure 4.11: Precast reinforced concrete walls
(Police Training College residential buildings, Porirua)



(c) General views
of precast walls of
first story



(d) View of top of
precast walls
before placing
cast-in-place floor



(e) A welded
connection in wall
flange above
cast-in-place floor

**Figure 4.11 (continued): Precast reinforced concrete walls
(Police Training College residential buildings, Porirua)**

The panels also provide vertical support for gravity load from the floors. A cast-in-place concrete foundation beam system gives support to the walls and provides resistance against overturning.

A relatively high seismic design load was chosen to reduce the ductility demand during a severe earthquake and prevent damage during moderate earthquakes. A horizontal dry-pack mortar joint was used between the floor slab and wall element above. Connections between vertical wall elements at the ends of the flanges and webs were made by welding the protruding reinforcing bars to steel plates (Figure 4.11(a)).

These connections were designed to transfer twice the design tensile force in the panel reinforcing and the associated shear, to ensure that any yielding would occur away from the connection. A cast-in-place concrete connection was made between the web and flange walls. Tests conducted in a laboratory on two panel units under cyclic loading in the yield range confirmed that a capacity design approach, in which the connections were deliberately made stronger than the elements connected, was essential if a reliable assessment of performance was to be made [4.19].

4.5.2 Rotorua District Council Civic Centre

This two-storey building utilizes full height precast concrete wall panels to provide support for the precast concrete floor system and the roof. The precast concrete walls, which provide the lateral load resistance of the building, are designed as cantilevered shear walls of limited ductility to the requirements of NZS 4203 [4.3]. The connection detail between walls and foundations was designed to withstand larger forces corresponding to an elastic response. Vertical joints between panels, shown during construction in Figure 4.12 consist of cast-in-place concrete joining horizontal overlapping reinforcing steel hair pins which project from each of the wall panels, creating a monolithic joint. A vertical steel bar was placed in the space between the ends of the hair pins prior to concreting. Details of the connection between the wall and the foundation are shown in Figures 4.13 and 4.14. Holes were formed in the bases of the panels so that horizontal reinforcing bars could be placed through to reinforce the joint. Due to the squat shape of the walls this detail was adequate to resist design horizontal shear forces and tension forces resulting from overturning moments.

4.5.3 Salvation Army Citadel, Vivian Street, Wellington

Four-storey high precast concrete walls occur on two

parallel sides of this building to provide seismic lateral load resistance in the north-south direction. These walls are simple cantilevered shear walls designed for limited ductility.

The walls are approximately 13 m high x 20 m long x 200 mm thick. The precast concrete wall panels used were two storeys high and 2.7 m long. The horizontal joints between the precast sections have lap bars grouted into metal ducting similar to the detail shown in Figure 4.10(b). These bars lap with pairs of vertical flexural bars in the precast panels. The vertical joints between panels as shown in Figures 4.15 and 4.18 have a half thickness cast-in-place concrete bandage joint with 90° hooked drop in lap bars.

A pair of large perforated tilt panel walls with a coupling “beam” tilt panel between, provide seismic load resistance in the east-west direction. This wall-beam system as shown in Figures 4.16 and 4.17 has been designed for limited ductility. The walls at 5.1 m wide by 7.5 m high with four large window openings behave more like deep membered frames than cantilevered shear walls. These tilt walls sit on a one-storey high cast-in-place concrete wall with a base connection detail similar to that shown in Figure 4.10(b). The

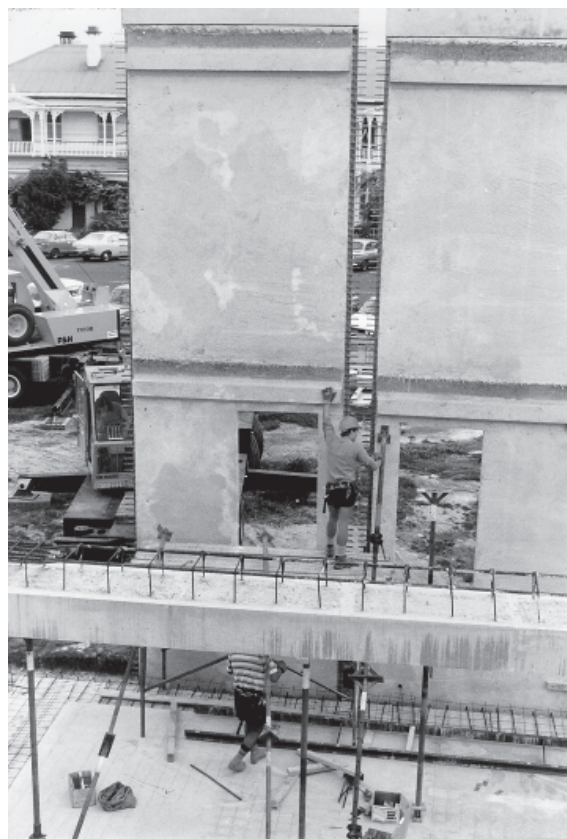


Figure 4.12: Precast concrete panels during erection (Rotorua District Council Civic Centre)

coupling “beam” tilt panel was connected to the adjacent panels by full strength butt welding 3-100 x 16 mm mild steel flats at each end of the beams. All the tilt panels were “match” cast to ensure a high degree of accuracy for the welded connections.

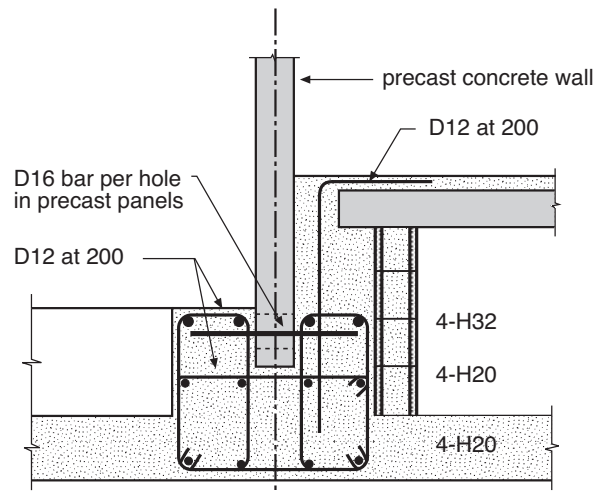
A simplified detail of the welded connection is shown in Figure 4.19.

4.5.4 Sheraton Hotel - Auckland

The main block of the Sheraton Hotel consists of a ten-storey building. Two cast-in-place walls are combined

with precast concrete walls to resist seismic loads in one direction while at right angles the total load is carried by precast concrete walls only.

The walls have been designed as ductile cantilever shear walls complying with NZS 4203:1976. The Structural Type Factor, S , used in the design was 1.0 and the Structural Material Factor, M , used was 1.0. A typical cross section of a precast wall unit is shown in Figure 4.20. The critical end regions of the precast walls are confined by stirrup cages as shown in that figure.



**Figure 4.13: Detail at the wall/foundation junction
(Rotorua District Council Civic Centre)**



**Figure 4.14: Connection between base of wall panel and foundation
(Rotorua District Council Civic Centre)**

The floors consist of precast untopped units, supported on the walls as illustrated in Figure 4.21. Continuity is established between the floor units by hairpin bars that overlap and are linked by a longitudinal bar in the joint placed above the precast wall unit.

The typical precast concrete wall units have 65 mm diameter ducts at 150 mm centres at the ends of the wall and at 300 mm centres in the middle regions. Deformed

reinforcing bars were placed in each duct as shown in Figure 4.21. These bars extend a lap length above the surface of the floor slab ready to receive the next precast wall unit. The vertical reinforcement placed in these ducts consists of 28 mm diameter bars in the end regions and 12 mm diameter bars in the mid regions. The duct together with the gap between the floor units above the precast concrete walls was fully grouted.

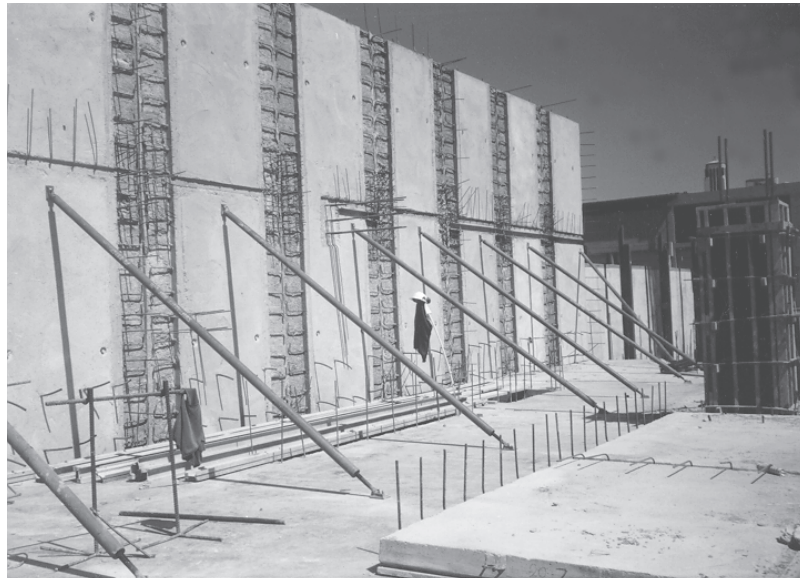


Figure 4.15: Interior view of part of 4-storey precast walls (Salvation Army Citadel, Wellington)



Figure 4.16: Interior view of perforated tilt panel walls showing location of weld connections (A) 2 storey perforated tilt panel walls, (B) Coupling “beam” tilt panel (Salvation Army Citadel, Wellington)



Figure 4.17: Perforated tilt panel walls (Salvation Army Citadel, Wellington)

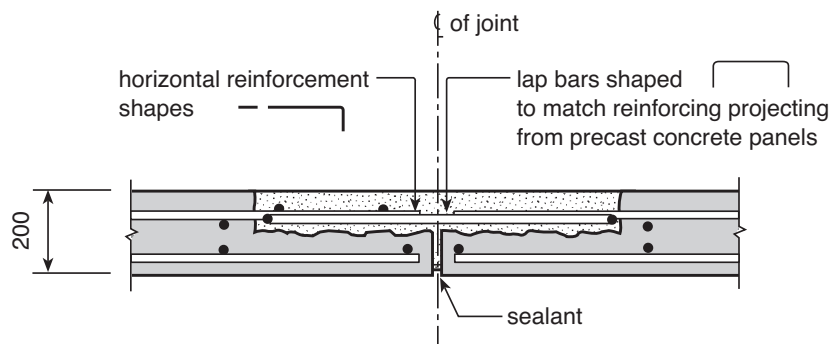


Figure 4.18: Vertical joint between precast concrete panels (Salvation Army Citadel, Wellington)

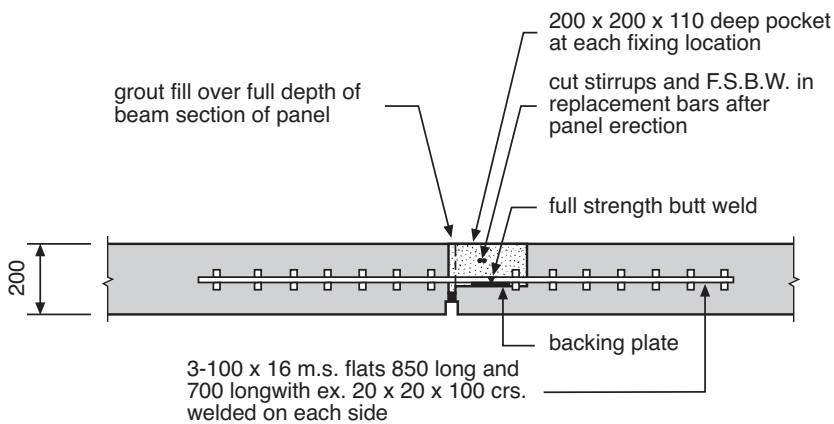
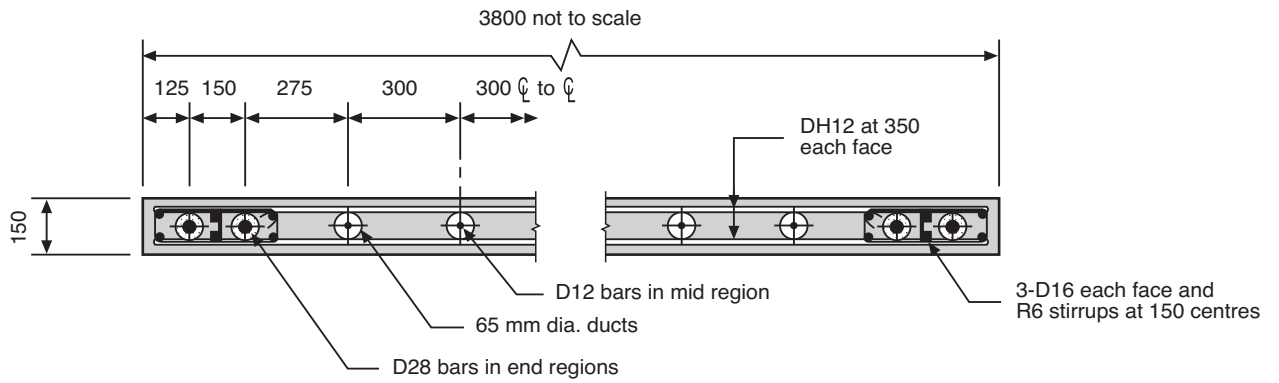
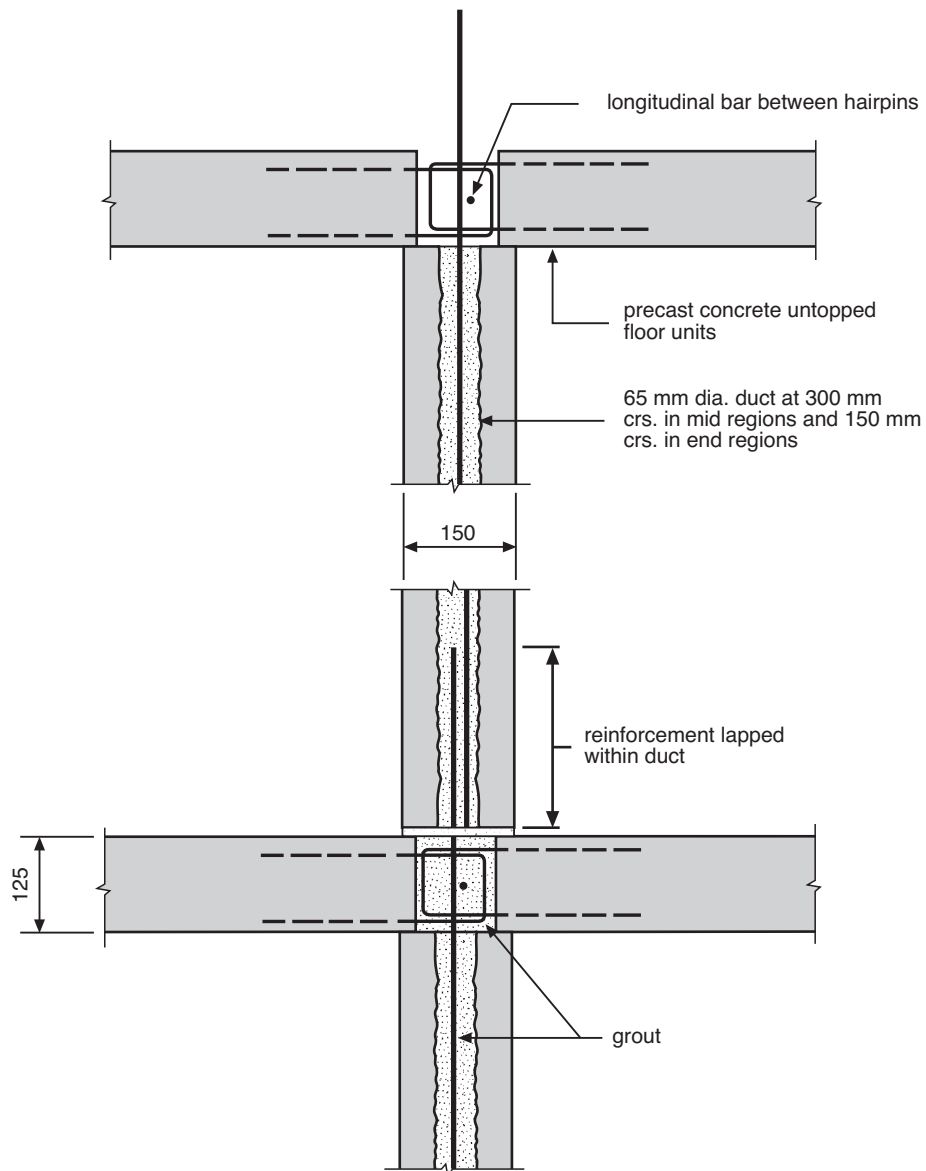


Figure 4.19: Connection between tilt walls and coupling beams (Salvation Army Citadel, Wellington)



**Figure 4.20: Horizontal section through typical precast wall panel
(Sheraton Hotel, Auckland)**



**Figure 4.21: Vertical section through precast wall - floor junction
(Sheraton Hotel, Auckland)**

4.6 Weathering Details

When precast concrete structural walls are used as part of the external envelope of a building, special attention to the joints is usually required to ensure weather-tight construction. During the life of a building small movements will often occur at wall panel joints through the actions of concrete shrinkage, creep, thermal movements, settlement or seismic movements. For exterior walls it is usually considered prudent to seal all junctions between adjacent precast concrete wall elements and all junctions between precast concrete wall elements and adjacent cast-in-place bandage joints with flexible sealants. Rebates are usually required at the junction between precast panels and cast-in-place bandage joints to accommodate the weatherproofing sealants. Often designers detail stepped or sloping horizontal wall panel joints to assist with weather proofing. These stepped or sloping horizontal joints are usually sealed with flexible sealants.

4.7 Recommendations

4.7.1 Monolithic Wall Systems

If the connections between precast concrete panels have been designed and detailed to possess stiffness, strength and ductility comparable to cast-in-place concrete construction, there is every reason to believe that a precast concrete wall would perform as satisfactorily as a cast-in-place wall. Care is needed with the detailing of both horizontal and vertical joints to ensure the satisfactory behaviour of the wall, particularly over the zone of yielding at the base of the walls (plastic hinge zone).

4.7.2 Jointed Systems

It is recommended that a very cautious approach be taken to the design of jointed precast concrete panel systems. Their seismic performance is clearly very different from that on which New Zealand codes of practice are based. A design approach based on both experimental test data verifying the seismic behaviour and detailed theoretical analysis is recommended [4.4].

4.7.3 Precast Concrete Wall Designs and NZS 3101

It is recommended that research is needed to check the sensitivity of monolithic precast concrete wall design construction to:

- the discontinuity at horizontal joint locations (Section 4.3.2);
- single-layer lap bar reinforcement at horizontal joints (Section 4.3.2);

- lap bars with diameters greater than one tenth of the wall thickness (Section 4.3.2);
- not complying with the full requirements for staggering laps in plastic hinge zones (Section 4.3.2); and
- concrete compression zones, in plastic hinge zones, of walls with single layers of reinforcement and no transverse reinforcement (confinement) at the ends of these wall regions.

4.8 References

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Chapter 5

Diaphragms

5.1 Introduction

5.1.1 Definition

A diaphragm in the context of this chapter is a horizontal or near horizontal element, such as a floor or a roof, which links the lateral force-resisting members in a building. In addition to distributing horizontal forces to the lateral force-resisting elements, it ties the structure together. Two forms of diaphragm have been identified [5.1]. The first of these is the simple diaphragm, which distributes forces that are applied directly to it to the lateral force-resisting system. These forces may arise from seismic inertial effects, wind or soil pressures. The second of these is the transfer diaphragm, which transfers shears arising from wind or earthquake actions between the different lateral force-resisting elements, in addition to functioning as a simple diaphragm.

5.1.2 General

The continuity of reinforcement, which is achieved in typical cast-in-place floor slabs, provides diaphragms with a high degree of integrity. With precast construction incorporating a cast-in-place topping, the level of integrity is reduced as only some of the reinforcement is continuous. The New Zealand Concrete Structures Standard gives requirements for the interface between cast-in-place concrete and precast components [5.2]. Where this interface is required to sustain cyclic forces arising from bars, which are subjected to yielding in tension and compression, caution is required. Tests with dry extruded members (hollow-core type components) have indicated that bond failure can occur between the precast and cast-in-place concrete at low shear stress levels [5.3]. This is not such a problem with units made by normal casting methods as the surface can be roughened and treated to remove laitance.

In the case of precast concrete flooring units without structural topping, continuity is generally provided by having relatively few connections between the precast units. This results in a loss of redundancy and greater vulnerability to damage from concrete volume changes and seismic actions. In this case particular care is required in assessing the forces and displacements imposed on the connections and ensuring these have adequate ductility to avoid premature brittle failure.

This chapter addresses the question of what characteristics precast concrete diaphragms require to perform adequately. The actions arising on these are discussed and recommendations for their design are made. In researching this topic a major difficulty was apparent. With many structural elements extensive testing has been carried out to check theoretical predictions and devise design criteria. However, with diaphragms the background of structural testing is very limited due to the difficulty of modelling the situation realistically and applying the high forces involved. As a result of the lack of this basic research, designers are urged to take a conservative approach to the design of floors that are required to act as diaphragms.

5.2 Requirements of Diaphragms

5.2.1 General

Most diaphragms serve a dual purpose in that, as well as tying the structure together and distributing lateral forces arising from wind and seismic actions, they are called upon to also resist gravity loads. In this chapter only the diaphragm actions are discussed, but the simultaneous gravity load actions must also be considered.

In the design of diaphragms the following requirements need to be considered:

- robustness of the structure;
- serviceability limit state; and
- ultimate limit state.

5.2.2 Serviceability Limit State

Combinations of actions, which may be expected to arise several times during the life of the structure, need to be considered to ensure it is serviceable. Further details are given in Section 5.4.3.

5.2.3 Ultimate Limit State

The design strength [the design strength is equal to the nominal strength (theoretical strength) times the strength reduction factor] of the structure is required to be equal

to or greater than the maximum design action (the design action is equal to the sum of the load factored actions) determined from the specified load combinations. As yielding of the reinforcement is permitted in this limit state, redistribution of actions due to plastic behaviour may be assumed and the forces (but not cracks) arising from internal restraint due to factors such as creep, shrinkage or temperature change, may be neglected.

For structures that are designed to be ductile in severe earthquakes there is a further requirement related to capacity design. This requires the nominal strength of a diaphragm to be sufficient to sustain the structural actions associated with over-strength actions acting in the potential plastic hinges of the chosen energy dissipation mechanism. Further details on the ultimate limit state requirements are given in Section 5.4.3.

5.2.4 Robustness

An important aspect in the design of precast construction is to ensure that the structure maintains its integrity in the event of premature failure of a connection or member. Following a progressive collapse in the corner of a multi-storey precast concrete building at Ronan Point in London in the 1960s, specific design requirements were introduced into the British code of practice for the structural use of concrete. These were extended and incorporated in the British code, BS8110 [5.4], where they are referred to as “robustness requirements”. In the 1995 revision of the New Zealand Concrete Structures Standard [5.2] the term “structural integrity” is used instead of robustness.

Premature failure of an element, such as a column or a beam, may arise for a reason not specifically considered in the design. Reasons for such failures may include: an intense localised fire, impact of vehicles, poor workmanship, explosions due to gas, flour milling or chemicals, or seismic effects not allowed for in the design, such as the impact with adjacent structures or falling material. In the event of such a failure it is important that a progressive collapse is prevented. The robustness requirements of the British code are summarised below. For details, the reader is referred to reference 5.4. In developing these criteria, seismic considerations did not receive the detailed attention that would be appropriate for seismically active regions.

- The key load bearing elements in a structure, that is those members whose removal or failure could lead to a general collapse, must be identified and specifically designed, or otherwise protected, to prevent removal or failure by accident (gas explosion, impact with vehicle, etc.).
- In the design, the removal in turn of each vertical load carrying element, excluding the key elements

identified above, is considered. Under this condition, collapse of a significant part of the structure must not result.

- All buildings should be capable of resisting a notional horizontal force, applied at each floor simultaneously, of 1.5% of the characteristic dead weight of the structure at that level.
- All diaphragms are to be provided with effective horizontal ties at each level:
 - (a) around the periphery;
 - (b) internally; and
 - (c) to all columns and walls.
- Each column, or other vertical load carrying member, is to be tied to each floor so that the tie can resist a force not less than:
 - (a) a value related to the loading and tributary area supported by each column or wall; and
 - (b) 3% of the design vertical load carried by the member.

In the 1995 edition of the New Zealand Concrete Structures Standard [5.2] a number of structural integrity requirements, which relate to the design of diaphragms, were introduced. These are contained in two groups of clauses, namely 4.3.6 and 13.4.3. In section 4.3.6 (see NZS3101, 1998 amendment) the designer is required to provide a rational load path for forces acting on a diaphragm. In addition, for the cases where the building has three or more storeys and it is supported on precast walls there are three requirements for nominal reinforcement as shown in Figure 5.1 and described in the following paragraphs.

- Reinforcement placed parallel to the precast units is required to tie units together above internal supports and to tie the units into the external walls. This reinforcement is to be proportioned to resist a force of not less than 22 kN/m and the spacing is not to exceed 3 m.
- Reinforcement transverse to the span of the precast units is also required to resist a force of 22 kN/m times the length of the span(s) attributed to the supporting wall, based on simply-supported spans. For example, an exterior wall, supporting a 10.0 m span of precast floor, requires $(10/2) \times 22 \text{ kN/m} = 110 \text{ kN}$ capacity, in the line of the supporting wall. In this case the reinforcement may be located either in the topping, or the supporting wall elements within 600 mm of the plane of the floor, or it may be divided between these zones. The tension capacity has to be continuous over the full width of

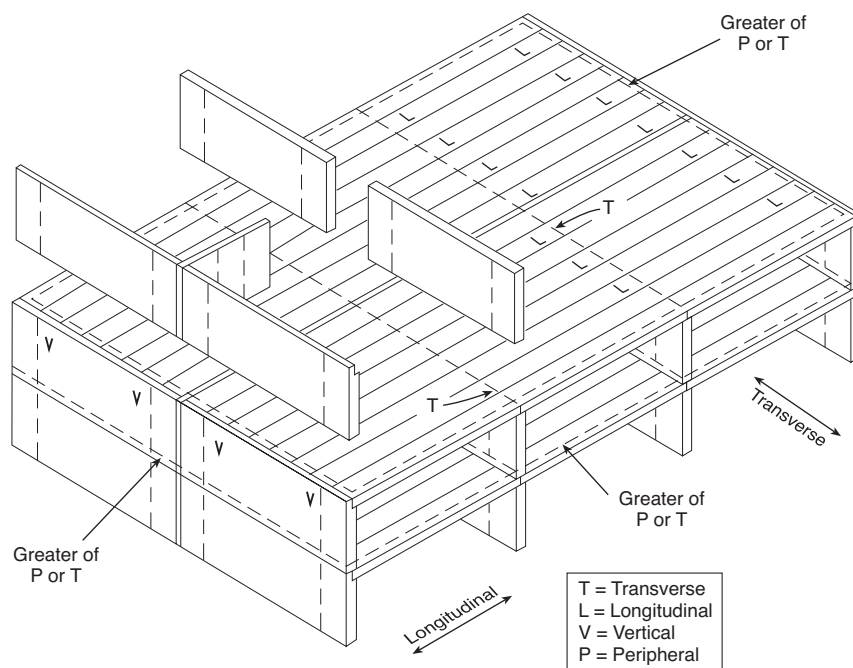


Figure 5.1: Typical locations for tying reinforcement in large panel structures [5.2]

the structure. There is no maximum spacing of reinforcement specified for this case.

- Reinforcement is required to provide a continuous tension capacity of not less than 70 kN right round the perimeter of the building. This reinforcement is to be located within 1.2 m of the building perimeter and it may be located within the perimeter walls, precast diaphragm units or topping.

The American Concrete Institute code (ACI 318-95) has very similar requirements as does the CEB-FIP model code [5.5], though in the latter code the requirements are very sensibly not limited to structures supported on precast walls. It is suggested that the NZS 3101-95 provisions could well be applied to structures supported on other components, particularly where masonry walls are used.

Robustness is an important characteristic for structures designed to behave in a ductile manner during severe earthquakes. Many actions that may occur in such an event tend to be neglected. These include the effects of vertical accelerations, which may increase or reduce gravity actions, differential vertical deflections due to surface waves and the elongation of beams, columns and walls due to the formation of plastic hinges. The detailed consideration of all possible effects and their combinations is impractical. To prevent possible collapse due to these actions robustness is required.

The elongation of members, such as beams, due to the formation of plastic hinge zones has received little

attention in the literature. However, as discussed in Chapter 2 and Appendix A1, it could, in extreme situations, lead to loss of support for precast concrete floor or stair systems. Numerous tests have indicated that, at a displacement ductility of 6, an elongation of 2 percent or more of the overall depth of the beam can be expected at each plastic hinge, see Appendix A1. This raises the possibility of one floor falling onto another and setting off a progressive collapse. This situation occurred in some parking buildings in the Northridge earthquake [5.6]. Provisions to prevent this occurrence are detailed in sections 2.3 and 2.4.

Additional potential elongation effects in ductile moment resisting frame structures, associated with the formation of plastic hinges in the beams, are outlined in the following paragraphs.

- Additional lateral deflection of the external columns occurs as indicated in Figure 5.2. This results in increased rotations of the plastic hinges at the base of the columns, and the formation of additional plastic hinges in the external columns near or adjacent to the level 1 beams. The increased column shear and plastic hinge rotations should be considered in design. This behaviour has been observed in a frame test [5.7].
- With precast floors beam elongation may be expected to generate wide cracks in the diaphragms at the sections which are weak in tension. These are at the support positions of the precast units, at the junction between any two precast units and be-

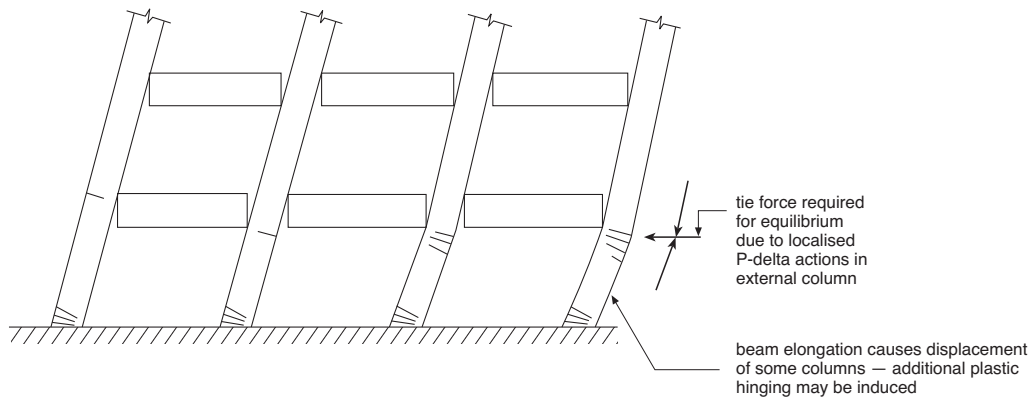


Figure 5.2: Localised P-delta action and additional hinging in columns due to beam elongation

tween a precast unit and a beam which is spanning parallel to the unit. In these locations the strength depends on the reinforcement in the cast-in-place concrete. Wide cracks result in a major reduction in both strength and stiffness of diaphragms. This is considered in sections 5.4.3 and 5.5.

- The high flexural and shear deformations in the plastic hinge zones impose high deformations on the precast units in their immediate vicinity. This may lead to local failure of non-ductile precast floor components, as discussed in section 5.5.
- The elongation of the plastic hinge zones can result in tension being induced in the precast units that are close to the plastic hinges. This is particularly critical if the plastic hinges are adjacent to the mid-span regions of the precast unit. The compression force balancing the additional tension force in the slab acts on the beam and it can significantly increase the resultant flexural strength. This has important implications for capacity design, as the design actions in the beam-column joints, shear in the beams and the strength of the columns are derived from the over-strength of the beam plastic hinge zones. This aspect is considered in more detail in section 5.5.

It should be noted that current methods of analysis, including time-history methods, do not predict the increased lateral displacements, column shears, plastic hinge rotations and additional column plastic hinge formation, which arise due to elongation.

5.3 Internal Restraint Actions in Diaphragms

5.3.1 Volume Changes

These arise due to creep, shrinkage and thermal effects in the concrete. Their main significance lies in their

influence on the serviceability of the structure. Differential creep and shrinkage effects in concrete members can lead to significant out-of-plane deflections. Floor slabs and roofs exposed to the sun can sustain appreciable deformation due to differential temperature conditions. As described in section 2.3.4 these can cause damage if adequate allowance is not made for them. Appendix C1 indicates how these actions can be assessed.

Volume change strains resulting from creep, shrinkage and temperature change (seasonal variation) can result in crack formation in the concrete topping. In some situations the formation of a crack, at say a control joint in the slab, may suppress the opening up of other control joints, leading to a few unacceptably wide cracks forming in the structure. A method of assessing potential crack locations and widths is outlined in Appendix C2.

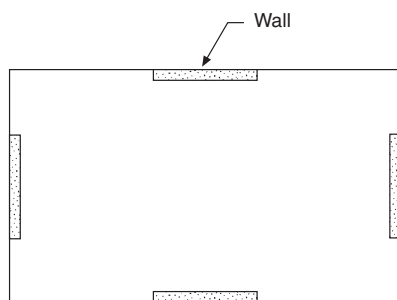
5.3.2 Minimising Volume Change Restraint Forces

Where practical, the lateral force-resisting elements in a structure should be arranged to minimise volume change restraint actions. For example, in the building shown in Figure 5.3, the arrangement of walls in (a) allows the volume changes to occur with a minimum of restraint, while the arrangement in (b) is likely to cause either extensive cracking in the suspended floors, or diagonal cracking in the walls.

5.4 Analysis and Design of Diaphragms

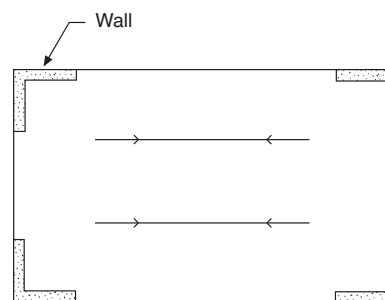
5.4.1 Diaphragm Flexibility

In New Zealand many simple diaphragms consist of precast concrete units with cast-in-place concrete topping. These are commonly assumed to be rigid in the structural analysis for seismic actions. However, in the



Plan

(a) Restraint to volume change
Strains minimised



Plan

(b) Walls restrain movements due to volume change (not preferred)
This arrangement may lead to the formation of wide cracks in the slab or diagonal cracking in the walls

Figure 5.3: Structural arrangement to minimise volume change restraint actions

transfer diaphragm case, where the level of shear sustained is high, it may be necessary to allow for diaphragm flexibility.

Some common structural analysis programs, such as older versions of ETABS, assume diaphragms are rigid. Where these programs are used, some allowance for the flexibility of diaphragms can, in some instances, be incorporated into the analysis by reducing the stiffness of the wall or frame elements to which the lateral forces are being transferred.

The importance of flexibility in a diaphragm on the force distribution in a structure changes with the limit-state being considered. In general terms it is much more significant in the serviceability than in the ultimate limit-state, where inelastic behaviour and consequential increase in the displacements of the lateral force-resisting elements occurs.

The New Zealand Concrete Structures Standard [5.2] contains guidance on when a diaphragm may be assumed to be rigid for the purposes of analysis. The commentary to clause 13.3.4 indicates the rigid diaphragm assumption may be made where the lateral deformation of the diaphragm is less than twice the average inter-storey drift in the relevant storey found in an elastic analysis for the design seismic forces at the ultimate limit state. This provision is similar to the corresponding recommendations in the 1997 UBC code and the proposed International Building code 2000 [5.8, 5.9].

For design purposes, a diaphragm may be assumed to act as a deep beam. Cracking in the topping concrete due to volume changes and beam elongation can reduce both the effective flexural and shear stiffnesses of

the diaphragm. To assess the stiffness appropriate for an analysis it should be noted that cracking is likely to exist in the cast-in-place concrete between precast concrete units and at the support positions. The stiffness can be expected to vary with the direction of the action. In most situations the analysis is not sensitive to the diaphragm stiffness and only in a relatively few cases is it necessary to assess this.

Where the “strut and tie” approach is used to design a diaphragm the basic truss may be used to assess the stiffness. In such calculations allowance should be made for tension stiffening of the concrete.

5.4.2 Influence of Cracking on Strength

Tests on the transfer of forces across cracks by aggregate interlock action have been made for crack widths up to and including 0.5 mm. The stiffness has been shown to decrease with increasing crack width [5.10, 5.11 and 5.12] and at some stage the interlock action must become ineffective. This limit has not been established. Tentative recommendations are made in the following paragraph.

For wide cracks (where aggregate interlock action is ineffective) some shear force can be resisted by kinking of the reinforcement. Hawkins and Mitchell found that longitudinal reinforcement, which was placed in the bottom of slabs to control punching shear failures, could work at an angle of 30° [5.13]. If this mechanism is assumed for design two points should be considered, namely:

- (i) the diameter and spacing of the reinforcement should be such that the concrete is not split, as illustrated in Figure 5.4; and

- (ii) strain in the reinforcement should not exceed the level corresponding to the maximum stress value for the reinforcement. If this value is exceeded it must be assumed that the bar has failed in tension. Plain bars should be used where wide cracks are anticipated as the poorer bond allows yielding to develop over a longer length.

5.4.3 Design Limit States

To ensure adequate performance a diaphragm needs to be proportioned so that it can satisfy the different limit states.

Serviceability Limit State

Under serviceability actions diaphragms should be proportioned so that crack control and deflection criteria are satisfied and the members remain in the elastic range, with the possible exception of allowing limited yielding of reinforcement at expansion joints or at supports where continuity moments have developed. With this limit state restraint forces arising from creep, shrinkage and thermal effects should be included [5.2].

Ultimate Limit State

There are two sets of requirements that arise from this limit state:

- The design strength of the diaphragm must be sufficient to sustain the combinations of load factored actions associated with gravity loads and lateral forces (arising from seismic effects, wind or soil forces). Yielding of the reinforcement dissipates forces arising from restraint to volume changes in the concrete. Consequently, these actions do not

need to be considered for this limit state. However, the possible effects on the load resistance due to cracks, which may have been opened up by these actions, should be considered.

- The diaphragm must be designed to satisfy capacity design requirements. These ensure that the ductile primary energy dissipating mechanism can be sustained in a severe earthquake. For a ductile structure the capacity design principles must be used, with the nominal strength of the diaphragm being equal to or greater than the actions sustained when over-strength actions act in the potential plastic hinge zones.

The diaphragm chapter of the New Zealand Concrete Standard (Chapter 13 of Reference 5.2) also requires that the diaphragms should generally remain elastic under the application of the capacity design forces. This condition is specified as it is difficult to detail these elements to behave in a ductile manner. However, it should be noted that inelastic deformation is inevitable where the supporting beams contain potential plastic hinge zones. Once plastic hinging occurs, elongation ensures extensive yielding develops in the diaphragm. Where diaphragms are supported by precast walls the Standard requires minimum levels of nominal reinforcement to be used (see section 5.2.4).

In the situation where high shear forces are sustained by a diaphragm, which has beams embedded in it that are expected to form plastic hinges in a major earthquake, consideration needs to be given to the change in behaviour associated with beam elongation. The wide cracks that may be imposed on a diaphragm in this situation, can cause gross changes in both its stiffness

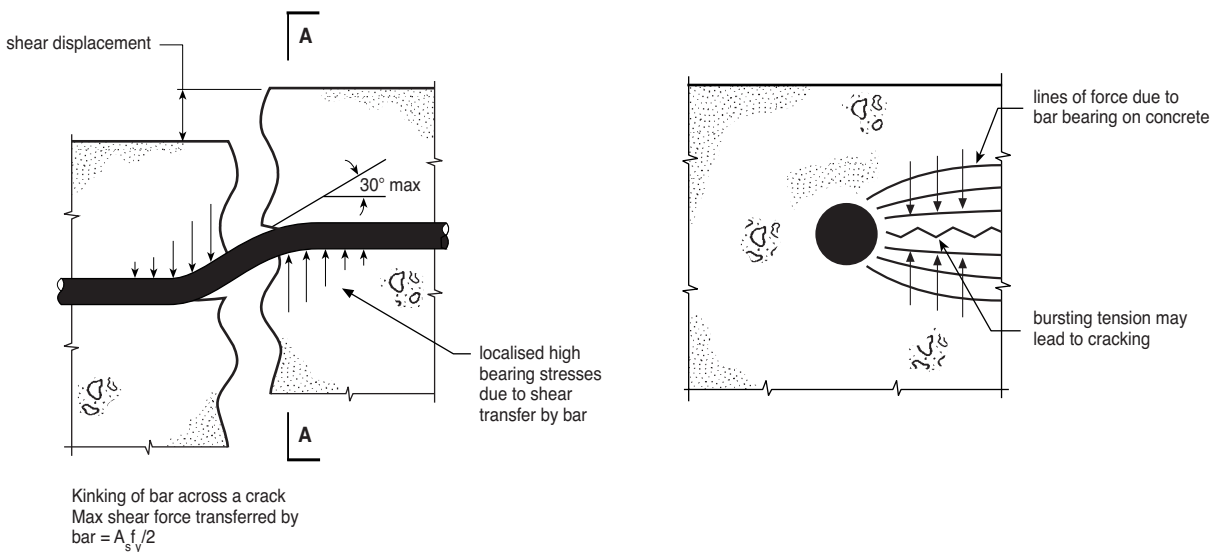


Figure 5.4: Shear transfer across a crack by kinking of bar

and strength. Consequently, the stiff behaviour of the diaphragm assumed in the elastic analysis of the structure could cease to be appropriate. In extreme cases, the diaphragm may start to distribute the seismic shears arising from loads it supports in a tributary mode. Consideration may need to be given to the change in the primary energy dissipating mechanism associated with the loss of stiffness and strength. Further research is required on the influence of beam elongation on the performance of diaphragms. However, an essential feature is to ensure that the vertical loading carrying capacity is not destroyed by elongation (see sections 2.3 and 2.4). Redistribution of seismic actions due to the loss of diaphragm stiffness can be accepted in the ultimate limit-state provided this does not lead to excessive inelastic deformations being imposed on the lateral force-resisting elements.

5.4.4 Strength of Diaphragms

Strut and tie method

This approach provides a valuable tool for the design of diaphragms [5.14, 5.15, 5.2 and Commentary to 5.2]. Intermediate beams in the slab can act in a similar manner to stirrups in a beam, with diagonal compression forces being resisted by the concrete topping and the precast units where these are continuous. A strut and tie model for a transfer diaphragm is illustrated in Figure 5.5. The need to provide reinforcement to transfer the forces to the lateral force resisting elements (such as wall or frames) should be noted. This is illustrated in the figure. The strut and tie method is particularly useful for complex details, such as may be caused by large openings in the floor [5.14]. The reinforcement in the topping may also act as tie or shear reinforcement. However, where there is poor bond between the precast units and the cast-in-place concrete, particular care is required to prevent delamination (see Delamination). This is likely to arise where a concentrated band of reinforcement in the topping is required to sustain high tensile stresses or where direct tension forces are introduced by reinforcement into the in situ concrete.

Delamination

Recent research has shown that considerable care is required where reinforcement located in the topping is used to transmit tension forces into a precast diaphragm. This is particularly a problem where the precast units are made from extruded dry concrete. The problem is accentuated where wide cracks develop due to elongation of the beams. Tests have shown that in some conditions delamination may occur at this interface. This arises as a result of either high bond forces generated by highly stressed reinforcement entering the

topping, or buckling of the reinforcement which has first yielded in tension and then subjected to compression, or a combination of these actions [5.3, 5.6].

The application of a tension force to the topping on a precast slab is illustrated in Figure 5.6, together with the failure mechanisms that have been observed. In these tests precast prestressed hollow-core units were used. These are manufactured from a dry concrete mix using an extrusion process. The units were eccentrically prestressed and they contained no vertical tie or stirrup reinforcement in the concrete between the cores. As shown in the figure, the tension force induced in the continuity reinforcement in the topping above the support, together with the balancing compression force, induced a couple bending moment in the concrete section at the end of the beam. Initially, a tension crack formed in the topping concrete close to the termination point of the starter or continuity bars. This was followed by a brittle failure of the wire mesh in the topping as it had a low ductility, and either a delamination failure or a failure in the reduced concrete thickness in the level of the cores.

The situation illustrated in Figure 5.6, where a tension force is transmitted to the topping over a wide crack, is improved by using plain bars. The reduced bond resistance allows yielding to extend further into the concrete and there is a reduction in the intensity of the delamination shear stresses. In addition there is less disruption of the topping concrete due to localised cracking associated with the high bond stresses of deformed reinforcement. The use of plain bars is particularly recommended where this reinforcement is expected to bridge wide cracks, such as may be induced by elongation of beams when plastic hinges are formed (see Chapter 2 and Appendix A1).

Where hollow-core units are used, the reliance on the bond between the in situ and precast concretes can be eliminated by breaking out the flange above two or more cores at the ends of each unit and adding reinforcement and cast-in-place concrete in the voids (see section 2.4.2).

Other factors

In designing diaphragms a number of other factors should be considered in connection with strut and tie analyses.

- High diagonal compression stresses in the concrete in diaphragms with hollow core units may cause horizontal splitting cracks to form as illustrated in Figure 5.7. Such cracking could allow the bottom half of a unit to drop out, leaving the top half with out appreciable flexural reinforcement to resist the gravity loads.

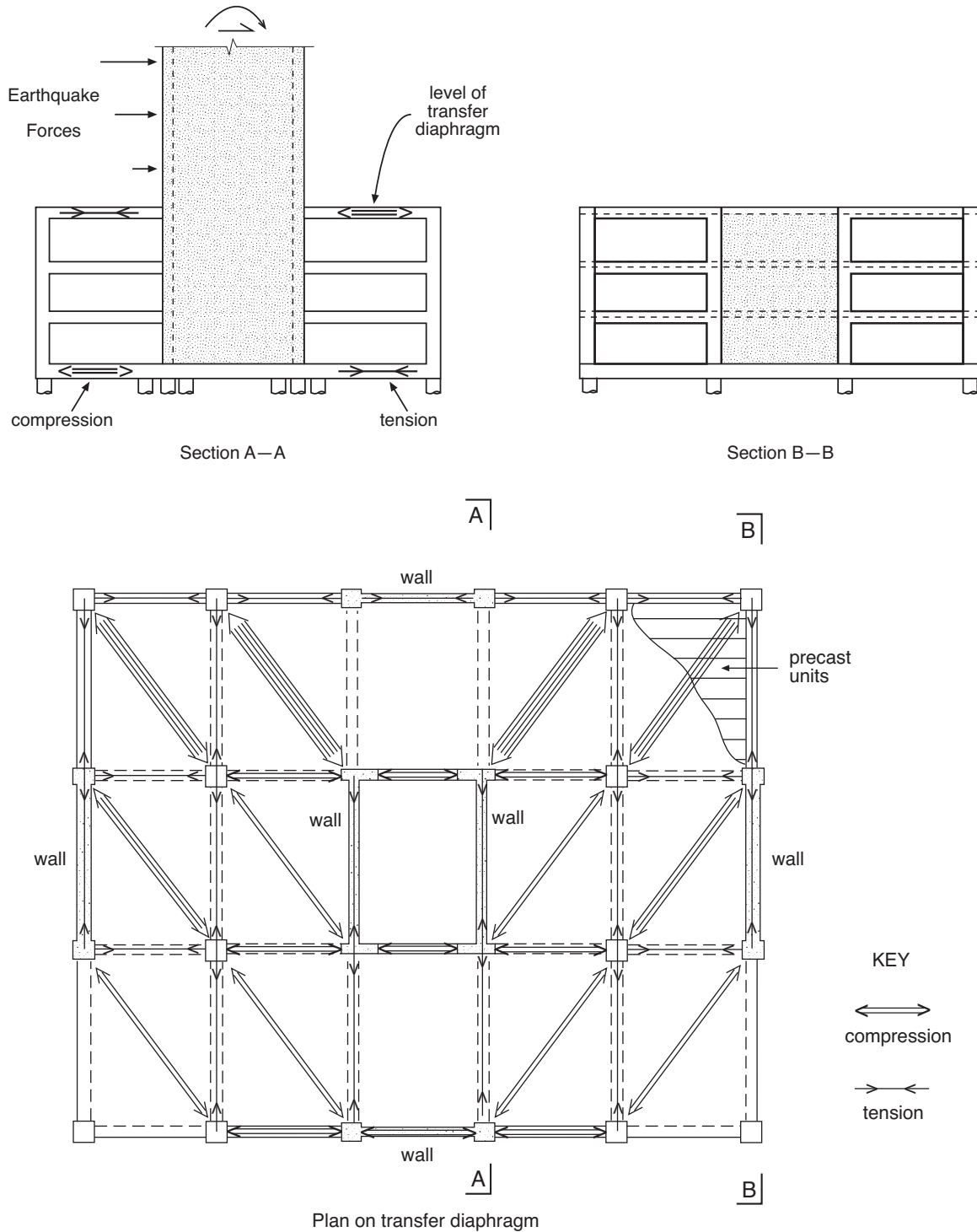


Figure 5.5: Action in a diaphragm — with strut and tie model

- There is a potential problem with diagonal compression buckling, which is particularly severe where there is poor bond between the precast and in situ concretes. This situation is made more critical when the in situ concrete contains significant reinforcement that has been yielded in tension and subsequently subjected to compression [5.3].
- Pretensioned reinforcement in the precast units

gives the diaphragm very different stiffness and strength characteristics in different directions. This aspect needs to be considered when the direction of struts and ties in strut and tie models is devised. The prestress inhibits the formation of cracking across the line of prestress, and this can have a direct effect on the formation of the actual struts and ties and hence the structural performance.

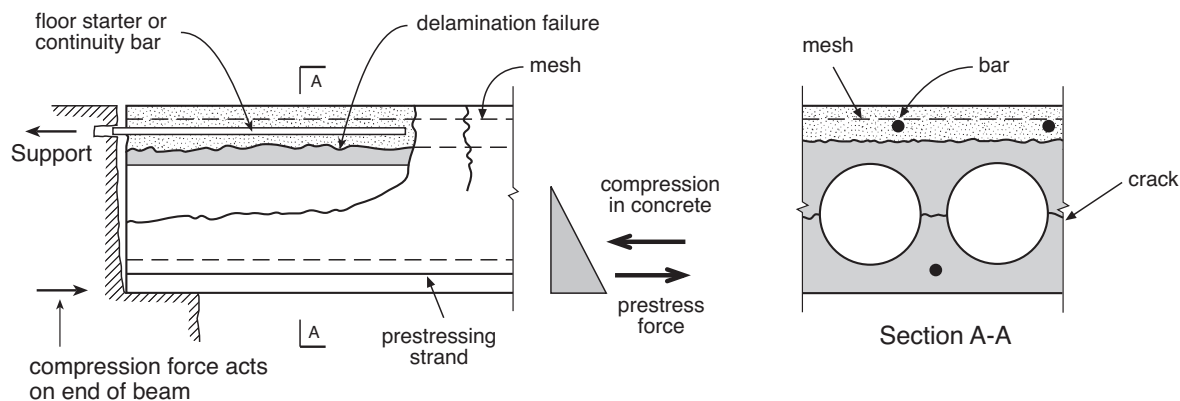


Figure 5.6: Potential failure mechanisms at the ends of hollow-core members with highly stressed bars in the topping

5.5 Interaction of Plastic Hinge Zones and Diaphragms

Numerous tests of beams and beam column sub assemblies have indicated that at design levels of ductility plastic hinge zones sustain both high elongation and high shear deformation in addition to flexural rotation. A typical average shear strain over a length equal to the beam depth is 0.03, while a typical elongation is 0.02 times the beam depth [see references A1 to A5 in Appendix A].

Where precast floor units are either placed adjacent to a plastic hinge, or supported by a beam containing a plastic hinge, high deformations may be imposed on the units. Where these are brittle, and particularly where they do not contain any shear reinforcement, the imposed deformation is likely to result in shear failure, as indicated in Figure 5.8. Research is required to establish the extent of this potential problem.

Elongation of beams, due to plastic hinge formation, may in part be restrained by diaphragm action. However, tests on beams with composite reinforced concrete slabs have indicated that these slabs provide no significant restraint. The reason for this is that once reinforcement in the slab yields, on load reversal it acts to prop open the cracks, and hence increase elongation [5.16 and 5.17]. Tests have indicated that very high axial forces were required to prevent elongation from occurring. If these restraint forces can be sustained the flexural strength of the beams is greatly enhanced [5.17]. For the purposes of capacity design it is important to assess the likely strength enhancement of the beams due to restraint from diaphragms. An under assessment could in some cases lead to a non-ductile failure occurring, possibly due to shear failure in beams or plastic hinges forming in the columns leading to a column sway failure mode.

Figure 5.9 shows a part of a perimeter frame for a

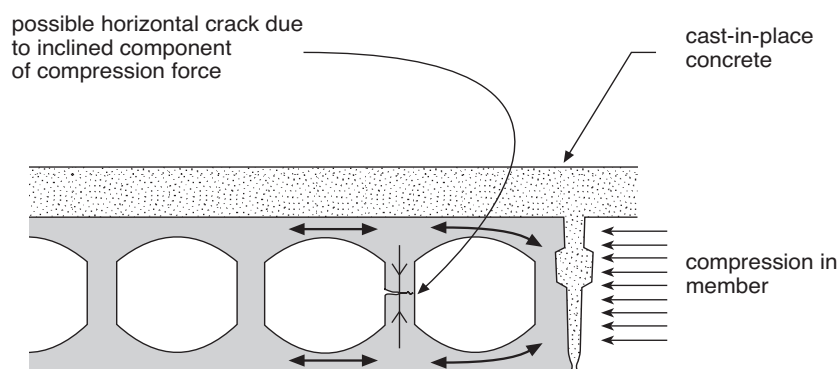


Figure 5.7: Possible horizontal cracking in hollow-core slabs due to compression

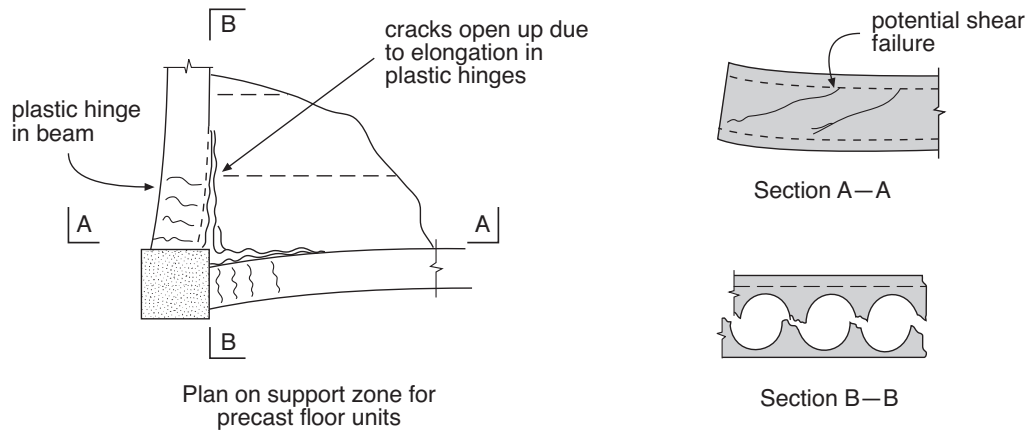


Figure 5.8: Deformations imposed on precast units adjacent to plastic hinges

building together with an associated diaphragm, made from precast pretensioned units and cast-in-situ concrete topping. With the arrangement shown in the figure the pretensioned units span directly past two perimeter columns, which are on gridlines 5 and 6. Elongation of the beams, due to plastic hinge formation at the faces of these columns, would be partially restrained by these units. There are a number of possible outcomes of this interaction between the beams and the diaphragm as indicated in the following paragraphs.

- The restraint provided by the diaphragm induces high shears at the interface with the beam, as illustrated in Figure 5.9 (b). This might lead to a complete shear failure at this interface or it might limit the shear transfer at this section. A situation similar to the one illustrated has been examined analytically in the literature [5.18]. The shear that could be transmitted across the critical interface was assessed with a strut and tie analysis (or by shear friction). It was found that this action increased beam negative moment flexural strength by 130 percent. There was no significant influence on the positive moment strength. However, the increase in bending moment resistance is sufficient to significantly raise the shear forces in the beams, and the moments and shears in the columns, which puts at risk the intended ductile failure mode.
- The resultant shear transmitted across the critical interface applies an eccentric axial tension force to the precast units. As illustrated in Figure 5.9(c) this may break the back of one or more units leaving it open to potential collapse when the tension force reduces.
- The restraint provided by the diaphragm is such that the axial compression force in the beams leads to a primary flexural compression failure. Such

failures would be associated with high strength gain in the beam.

A further source of strength increase due to diaphragm restraint to beam elongation needs to be considered. The transverse beam at column 4 supports the precast units. Elongation of the perimeter frame beams in the vicinity of this column could be expected to generate wide cracks in the topping on each side of the transverse beam. The situation is shown in Figure 5.9(d). Forces are transmitted across these cracks by reinforcement placed in the in situ concrete. However, in addition it can be seen that the two halves of the floor are acting as deep beams, with bending moments and shears being sustained at the end of the cantilever like members, see Figure 5.9(d)). This bending action, together with the force transmitted across the cracks along the transverse beam on line 4, applies an axial force to the beams in the vicinity of column 4, and this may substantially increase the flexural strength of the beams [5.18].

5.6 Overseas Practice

In the USA, with its wide range of seismicity, there are a wide variety of diaphragm construction practices. Generally there appears to be a preference for untopped precast concrete floor units to be used to form diaphragms. For example, the *PCI Manual for the Design of Hollow-Core Slabs* [5.19] gives information on the design of untopped diaphragms. Shear forces are resisted by shear friction along the longitudinal joints with the clamping force provided by transverse bars placed at the ends of the units. Limited full-scale testing [5.20] has shown that within the limits of the shear capacity available using this method, the system exhibited reasonable strength and ductility.

Untopped diaphragms usually contain embedded steel

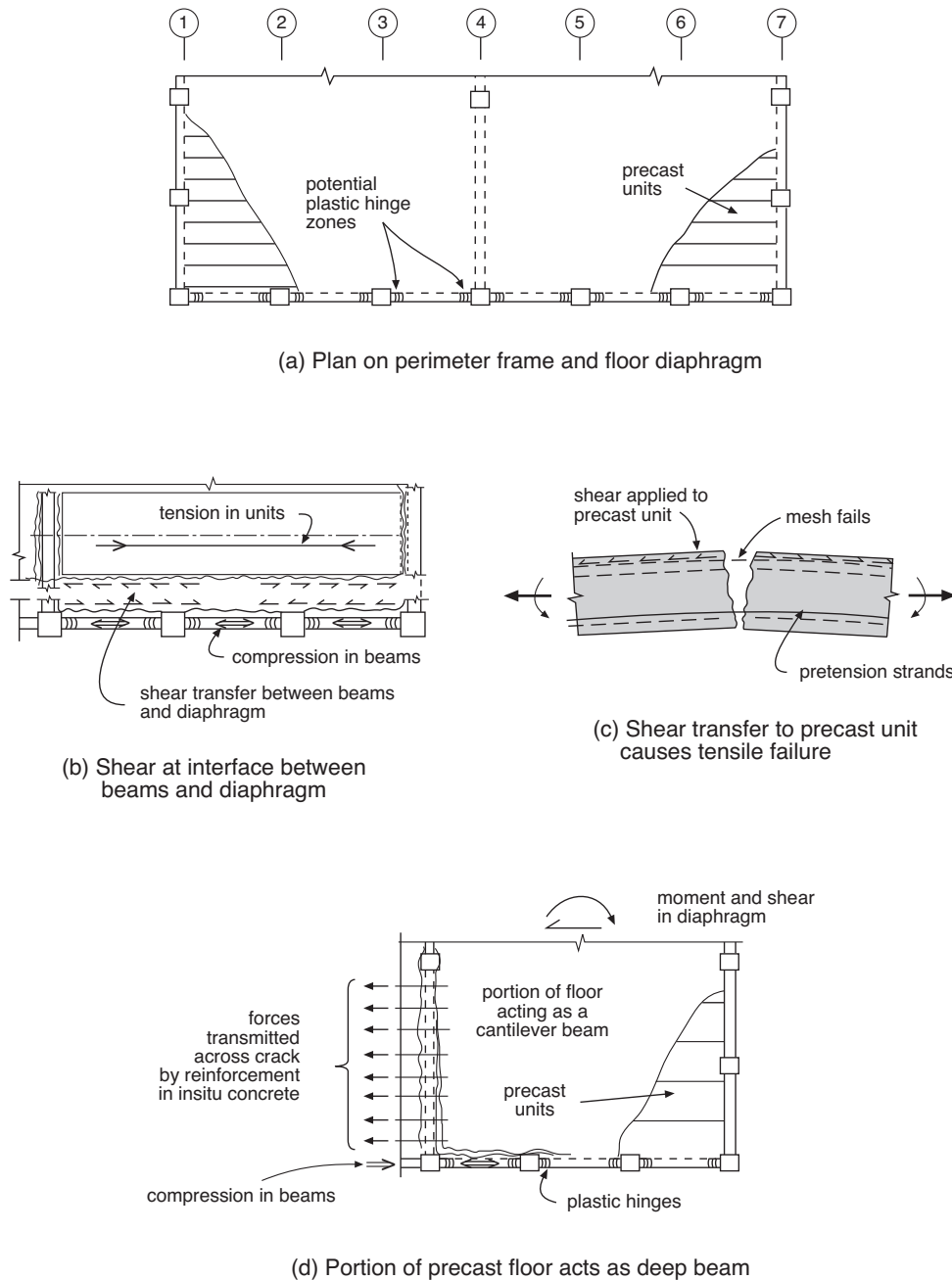


Figure 5.9: Interaction of beam elongation due to plastic hinges and diaphragm

plates, that are later connected by welding additional steel elements to these plates. The robustness of these connections is in doubt where they are called upon to resist seismic actions. Some testing of these connections has been carried out [5.21] and it was found that, provided that the loose plates, which were welded to steel embedments in the precast concrete units were slotted or narrow (to avoid brittle behaviour as the result of volume changes), the behaviour was satisfactory. Unfortunately only monotonic loading was applied so the results cannot be immediately adopted for diaphragms designed to resist seismic forces.

Concern has been expressed by several US engineers as to the adequacy of mechanical connections for diaphragms in seismic conditions [5.22, 5.23]. Clough has stated [5.25] that:

Untopped diaphragms in which inter-element connection is made by grouting or mechanical connectors, have relatively low in-plane shear strength and ductility and are most suitable when seismic equilibrium and compatibility forces are small. In zones of high seis-

mic intensity, or with structural configurations which impose large in-plane compatibility forces under lateral load, diaphragms joined by cast-in-place reinforced concrete, either as pour strip or as a topping, usually are more satisfactory.

From this brief survey of US practice some major concerns emerge regarding the use of untopped diaphragms in seismic situations.

European practice, at least as represented by the FIP [5.25], suggests an approach similar to that recommended by the PCI. The use of structural toppings in Europe is not common. However, in another FIP publication [5.26] structural toppings are acknowledged as improving diaphragm action in seismic conditions.

A review of Japanese practice for multi-storey frame buildings [5.27] indicates that cast-in-place concrete floor units tend to be used for more cellular construction such as apartment buildings [5.28]. The use of new systems in Japan is preceded by extensive testing programmes [5.27].

5.7 Recommendations

5.7.1 General

In this chapter a number of recommendations are made regarding the design of precast diaphragms together with areas that require further research.

5.7.2 Robustness

Satisfactory behaviour in extreme events, such as gas explosions, impact by vehicles or severe earthquakes, requires the structure to be robust. For this purpose appreciable structural redundancy is desirable so that the failure of one element does not lead to a progressive collapse. Where this is not practical the critical elements should be designed conservatively. In addition it is important that the different elements in a structure are tied together. In this regard attention is drawn to the “robustness requirements” in the British code and the “structural integrity requirements” in the New Zealand Concrete Standard (see section 5.2.4).

5.7.3 Internal restraint actions

Stresses and cracking may be induced due to internal restraint to movements induced by thermal effects, shrinkage and creep of the concrete. As shown in Section 2.3.4 considerable damage can occur if inadequate allowance is made for these effects. Methods of assessing thermal, shrinkage and creep actions and

associated crack widths, are given in Appendices C1 and C2. These effects are important for the serviceability limit state but of less significance for the ultimate limit state (see sections 5.3 and 5.4.3).

5.7.4 Analysis and Design

The influence of the flexibility of transfer diaphragms should be checked for the serviceability limit state. Criteria are given that indicate when allowance needs to be made for diaphragm flexibility and when the diaphragms may be considered to be rigid (see section 5.4.1).

Section 5.4.4 illustrates how the “strut and tie” method of analysis may be used to design a diaphragm. The importance of providing tie or drag bars to transfer the seismic force in the diaphragm into the lateral force resisting frames or walls is indicated. Caution is required where reinforcement is used to transmit tension direct into the in situ topping, as tests have shown that delamination can occur with some precast units. This is a particular problem at the support positions for precast units, where elongation due to the formation of plastic hinges in beams may generate wide cracks.

Where forces need to be transmitted across wide cracks (due to beam elongation) the situation is improved by using plain bars. The lower bond strengths of these bars reduces the delamination shear stresses and allows yielding to occur over a longer length giving an increase in ductility. With hollow-core units the tension force may be introduced into the ends of the units by locally breaking out the top flanges and placing reinforcement and in situ concrete in the voids (see sections 5.4.4, 2.3.5 and 2.4).

5.7.5 Deformation in Plastic Hinges

Wide cracks, associated with plastic hinge formation in beams embedded in a diaphragm, may greatly reduce the strength and stiffness of a diaphragm, changing the way it distributes forces to the lateral force resisting elements. The influence that this may have on the seismic performance of a building should be assessed. Where this may occur, designers should ensure that an adequate load path remains for the seismic forces (see sections 5.4.2 and 5.4.4).

The elongation in plastic hinges, in some situations, may be partially restrained by diaphragms, particularly where these contain prestressed units. This restraint can impose appreciable axial compression on the beam plastic hinges and result in very significant increases in flexural strength. Two situations where this may occur in perimeter frame beams are outlined. It is important to recognise and allow for this action as the unexpected over-strength could result in the formation of a non-

ductile failure mode, leading to premature collapse of the building, see section 5.5.

The high flexural and shear strains sustained by plastic hinges in beams embedded in a diaphragm may impose high deformations on precast units. As some of the commonly used units are relatively brittle in terms of the shear and flexural deformations, the possibility exists of local failures occurring in these units in the neighbourhood of the plastic hinge zones (see section 5.5). Research is required to establish the extent of this problem.

5.7.5 Untopped Diaphragms

In view of overseas findings untopped precast concrete diaphragms should not be used in a seismic resisting structure unless a special study is undertaken which includes assessing the effects of concrete volume changes. Particular care is required where this form of construction is used in frame structures, in which the beam may be subjected to elongation in a severe earthquake due to plastic hinge formation. Special detailing is warranted to ensure that ductile performance can be obtained (see section 5.6).

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Chapter 6

Connections between Precast Concrete Units by Grouted and Welded Bars

6.1 General

The grouting of bars and interfaces, and welding of reinforcing steel at connections between precast concrete structural elements, is common in the New Zealand precast concrete industry. Many types of connection, such as between columns and footings, which formerly would have always been of cast-in-place construction, now frequently rely on grouted connections for their structural integrity.

The basic requirement of any grouted or welded connection is that it satisfactorily meets all the strength, stiffness and other performance criteria for the serviceability and ultimate limit states. In most cases this means that connections should be at least as satisfactory in all respects as cast-in-place connections.

The design approach to grouted and welded connections is similar to that of cast-in-place connections. However, a higher level of workmanship and quality assurance is required. The procedures for making grouted and welded connections are far more complex, more difficult (sometimes impossible) to inspect, and it may not be possible to make adjustments to the connection after it has been made. High quality workmanship is essential to the satisfactory achievement of these connections.

In this chapter, guidance and recommendations are given on the different stages of designing and constructing grouted and welded connections.

6.2 Selection and Types of Grout

6.2.1 General Principles

Every grouting situation requires careful consideration by the specifier as to the most appropriate grout for the purpose.

There are a large number of factors that may influence the selection of a grout. These include the rate of strength gain, required final strength, shrinkage (or volume change) characteristics, long-term appearance, durability (under chemical and water attack), place-

ment conditions (presence of moisture or dust, hot or cold ambient temperatures, vertical or horizontal placement, etc.), viscosity, and “wetting” characteristics. Viscosity and “wetting” determine how effectively air is displaced and how well grout bonds to components of the assembly. These factors also dictate what clearance is needed between interfaces of assemblies or between duct walls and reinforcing bars. Also affected by grout type is the diameter of the tremie or injection tube used to place the grout.

6.2.2 Types of Grouts

Epoxy and Polyester Grouts

Epoxies and polyesters are two-part mixtures. They generally develop very high compressive strengths in a relative short time (40 MPa in 6 hours) and require shorter embedment lengths for grouted-in bars etc., than for cement-based grouts. However, the designer must exercise caution to ensure that even though the bond between grout, bar and surrounding concrete is sufficient, the failure mechanism does not become a concrete cone pull-out (see Chapter 7). Bar yielding followed by bar ultimate tensile failure is the preferred failure mechanism.

These grouts, which come in a wide range of viscosities and strengths, are sensitive to construction conditions. In particular, water and dust in a void can seriously reduce the strength as well as the bonding capability of the grout. Water can have a detrimental influence on the long-term properties of some polyesters.

The chemical reaction of these grouts is usually fast and generates significant amounts of heat. The resin should not be present in such a large volume that the heat of reaction causes the resin to boil. If this occurs the grout loses strength and becomes a granular unbonded mass.

The manufacturers’ instructions should be consulted for full details on appropriate usage of such special grouts.

Cement-based Grouts

Cement-based grouts, which are usually cheaper than resin grouts, are often favoured when it is not necessary

to use the rapid strength-gain of resin grouts. The typical strength development of cement grouts is 15 MPa at 1 day, reaching 60 MPa at 28 days.

Although in some situations neat cement grout can be used for grouting, high-strength, non-shrink cement-based grouts are usually specified. High strength grout is specified, firstly to ensure that forces in the bars can be transferred to the surrounding concrete and, secondly, to minimize the time before the assembly can withstand loading.

It is recommended that the minimum grout strength be 10 MPa greater than that of the surrounding concrete. Non-shrink or expanding grout is specified to ensure that the bond and load carrying ability of the grout is not diminished by the presence of shrinkage cracks or entrapped air [6.1].

The manufacturers' instructions should be followed in detail when using proprietary grouts.

Occasionally, specifiers/contractors may choose to produce their own cement-based grout, accepting the responsibility for failure, rather than use a proprietary grout. In these circumstances there should be enhanced quality control of production, placement and testing (for verification of design performance) when compared to that required for proprietary systems.

6.3 Grouting Situations

6.3.1 General

In any grouting situation the following fundamental principle applies:

- Replace air in the void completely by grout, so that the full dependable load carrying ability and stiffness of the connection are achieved.

Situations where bars and interfaces are grouted generally fall into two groups, namely vertical and horizontal void grouting. A combination of these two situations within a particular connection detail is not uncommon. Techniques for successful grouting vary accordingly.

Bars and holes must be free of grease, paint, dust, rust and mill scale for the concrete/grout/steel bond to be fully effective. It is essential that surfaces are not oil contaminated from blowing out with compressed air containing oil (Section 6.6.2).

Loss of Concrete Cover due to Gap Sealing Materials

Perimeter gaps or ducts are sealed to prevent grout loss

during grouting by flexible spongy strips, epoxy putty, or cement mortars. Designers must allow for the possible reduction of cover to steel reinforcement caused by the use of sealing materials. Cover reduction may promote premature corrosion, and in these cases additional cover to such steel elements should be provided. A further consideration of the reduction of cover concrete is the shortening of the internal lever arm 'd' between the tension reinforcement and the concrete compression zone. The reduction of 'd' will decrease the available bending moment capacity at that section.

6.3.2 Vertical Void Grouting

Vertical Starter Bars

Starter bars can be grouted into vertical holes that can be either drilled, formed with formers left in place, or in formed holes with the formers removed (see Figure 6.1).

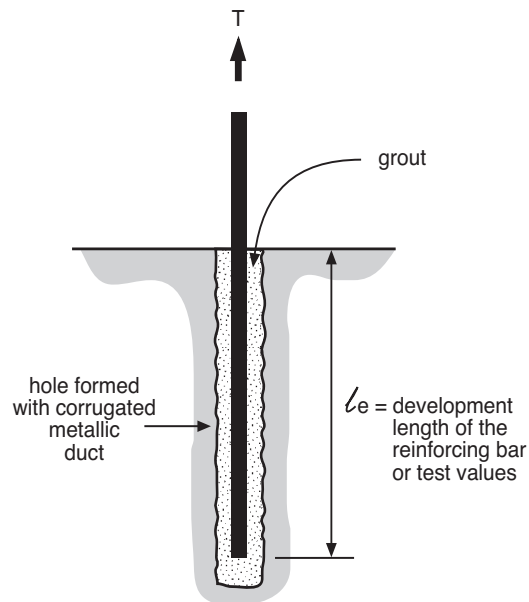


Figure 6.1: Preferred detail for formed voids

Masonry drills usually provide a desirable rough wall surface for good mechanical bond at the grout/concrete interface. However, care must be taken that the drilling method does not produce cracking in the concrete around the drilled hole which, although not readily apparent, could reduce the grout/concrete bond strength. Diamond drills will also generally produce a suitable surface for bonding. However, a polished surface, occasionally produced by diamond drills, may be detrimental to bonding capability and additional surface roughening can be required. This roughening may be done by mechanical scabbling of the wall of the hole. Some broken-in diamond drills produce a "rifling" effect that should produce adequate roughening. All dust and debris that can reduce the bond between the grout and concrete must be removed.

Formed voids should have some sort of roughening or corrugations to assist the shear stress transfer between the grout and concrete as illustrated in Figure 6.1. Where formers remain, experience and tests have shown that corrugated metallic ducts or conduits should be used. PVC, rubber or polystyrene formers do not perform satisfactorily when left in place.

As noted previously, the size of spaces or clearances between bars and holes is dependent on the viscosity and “wetting” ability of the grout. Low-viscosity, high-wetting grouts such as epoxies usually require only small clearances.

Placement of grout can be either before (pre-grouting) or after (post-grouting) the bar is located in the void. When the position of the bar is considered critical, the bar is usually located prior to grouting.

When pre-grouting, the hole is filled with sufficient grout so that it fills the hole completely as it is displaced by the bar. When clearances between the bar and sides of the hole are small, say less than 3 mm, it is recommended that a less viscous epoxy-type grout be used. Pre-grouting eliminates the critical workmanship phase of post-grouting and is easier and less time consuming.

For post-grouting, larger hole diameters are required than for pre-grouting. After the bar has been located in the hole, two grouting methods are suggested.

“Tremie Tube”

The tremie tube method, shown in Figure 6.2, is recommended. This technique is the most likely to produce a fully-grouted connection.

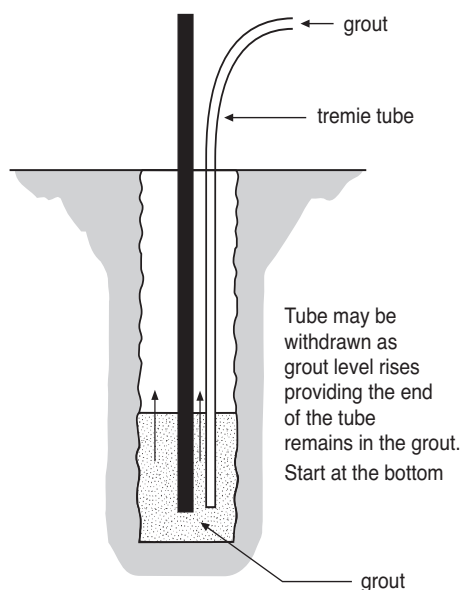


Figure 6.2: Tremie tube method of grouting

“Decanting”

If the clearances between bar and hole sides are sufficient then the grout may be poured against and down the bar to the bottom of the void. This method is susceptible to air-locks which may lead to impaired load transfer performance and corrosion. Air-locks result from either too high a rate of grout placement, a grout that is too viscous, or a combination of these effects.

Vertical Voids or Spaces

Figure 6.3 shows two typical situations of vertical voids to be grouted. The steel section and the precast concrete beam are grouted in the same manner. The width of the space to be grouted is usually set by construction tolerances and typically ranges from 20 mm to 50 mm. The sides and bottom of the space can be formed either by formwork (usually timber), or by sealing with a putty-like epoxy or mortar.

The grout used for such spaces is usually cement-based. The grout may be poured in from the top of the void provided the space is wide enough to prevent air-locks from developing. Experience and manufacturer’s recommendations will guide that decision. If the space is narrow and deep then it should be treated as a hole receiving a vertical starter bar (see Section 6.3.2.1) and precautions to prevent air-locks forming need to be taken. If the grout is expected to transfer loads its minimum strength should be specified. Where loads are to be transferred across the grouted connection, attention to details such as clean interfaces and appropriate roughening are essential.

6.3.3 Horizontal Void Grouting

Horizontal Starter Bars

A typical situation is shown in Figure 6.4. This type of detail is used where the presence of protruding reinforcing steel will hinder the installation or removal of formwork such as structural wall climbing forms. By casting in ducts at starter bar locations, the forms can be left free of starter bars which are grouted-in later.

Voids can be drilled or formed as for vertical starter bars. The “horizontal” nature of the voids creates the potential for air-locks at the tops of the spaces. Inclining the duct or hole from the horizontal allows the grout to displace the air as illustrated in Figure 6.5. However an allowance for additional embedment length must be made to account for the incomplete filling of the hole or duct (Figure 6.5(a)), unless it can be fully filled by sealing the end and grout pumped in so that it overflows out the top (Figure 6.5(b)). Any air gap (Figure 6.5(a)) should be filled with mortar or epoxy (putty consistency).

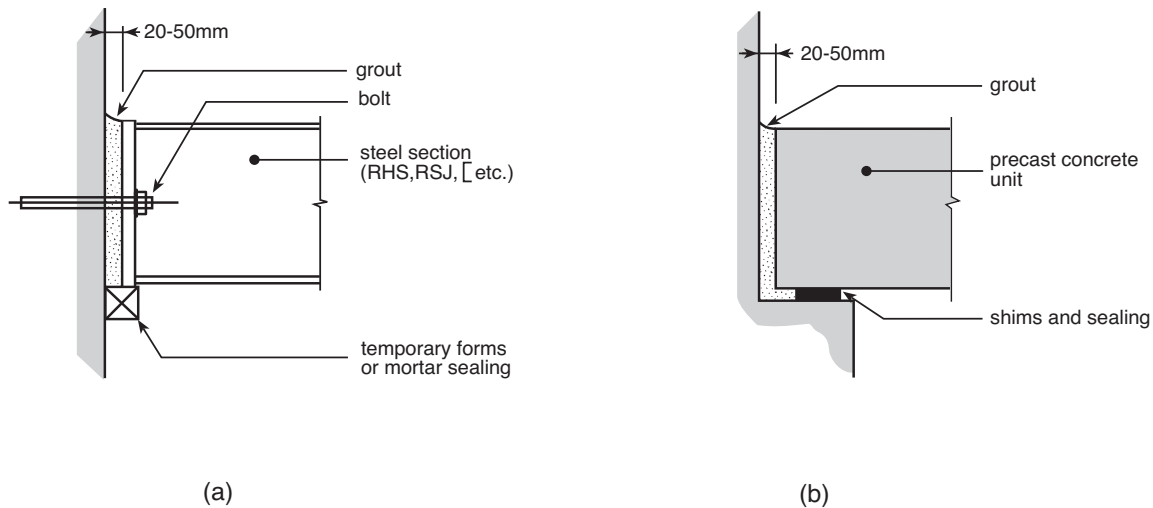


Figure 6.3: Vertical voids requiring grouting

Inclined holes, formed by ducts or drilling, tend to clash with other structural reinforcement. Care needs to be exercised with these details. Designers need to consider the bursting force generated at the bend in the reinforcing bar under tension (Figure 6.5(c)).

Where horizontal ducts are used, pressure grouting, similar to post-tensioning grouting techniques used in prestressing, can be employed to minimise the residual air as shown in Figure 6.6.

In detailing grouted bars, attention needs to be given to locating the bar in the centre of the hole. In particular, the bar must not end up in the top portion of the duct because of the potential for developing an air-pocket in this location (Figure 6.7). Bar locators should be used to place the bar in the centre of the duct and allow an unobstructed flow of grout throughout the duct.

When using horizontally grouted bars, designers should increase the embedment/lap length of the bar to allow for incomplete contact of the grout with the sides of the

hole/duct. A 20% increase in length is currently being specified by a number of designers. Notwithstanding this, every care should be taken in construction to avoid air entrapment and, as far as possible, achieve complete grout-to-hole/duct contact.

As with vertical starter bars, the size of the duct depends on restrictions imposed by the structure and its reinforcement as well as the type of grout to be used. Larger voids allow a greater tolerance in placing bars, but more attention must be given to rigidly supporting the bars during and after the grouting operation.

Horizontal Voids or Spaces

Unless special care is taken when grouting horizontal voids there is a tendency to trap air. Vent holes through the base plates of steel columns or machinery assist in relieving the trapped air. However, vents are usually not feasible where precast concrete units are located above a void.

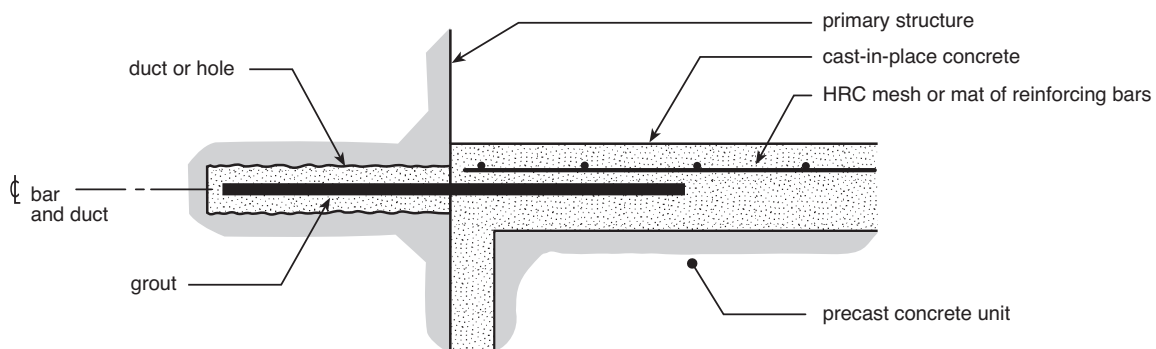


Figure 6.4: Typical horizontal starters

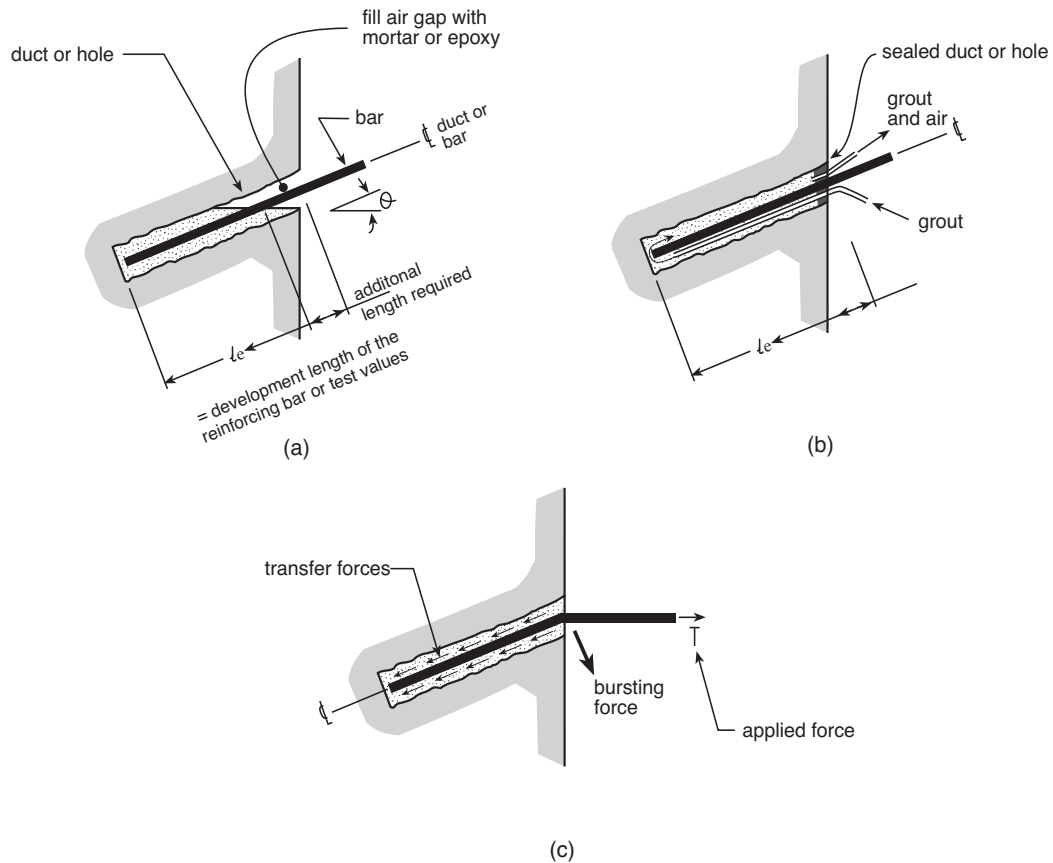


Figure 6.5: Inclined starters

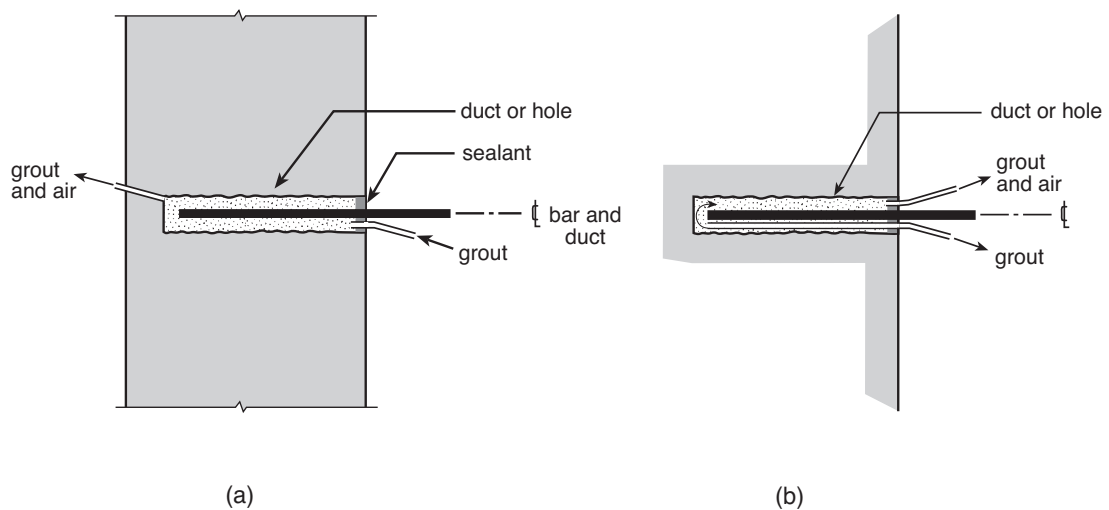


Figure 6.6: Grouting of horizontal starters

To overcome this problem, experience of designers and grouting specialists has shown that the grout (epoxy or cement-based) should be injected under pressure from one point or side (Figure 6.8). In the case of long walls (Figure 6.8(a)), timber bunds may be used to provide a head for the grout. For grouting under plates or precast units, the sides of voids are usually sealed by

a cement or epoxy-based mortar with the consistency of putty, which can be colour matched if necessary. As an alternative, compressible proprietary sealing strips can be used around the perimeter. Any sealing strip or compound will occupy some of the area of the joint which will reduce the bearing area or cross-sectional area of the joint. The designer will need to check that

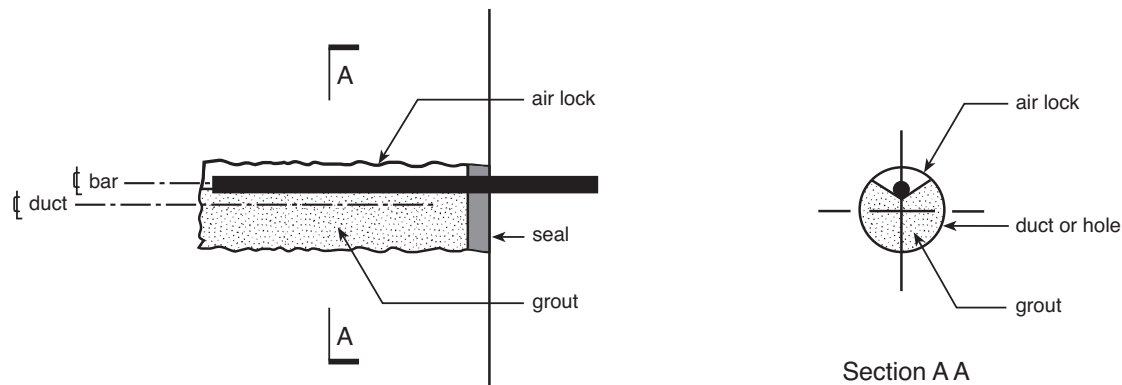


Figure 6.7: Bond loss in an air-lock

this reduction is not critical for the expected loadings and that cover to reinforcing steel is still adequate (Section 6.3.1.1). Outlet ports or tubes (Figure 6.8(b)) should be located so that air can escape from the void. When grout, free of air bubbles, is flowing from an outlet, the port should be plugged.

When the underside surface of the top unit has been roughened for shear transfer requirements, grout selection is particularly important. There is concern among some designers that fine air bubbles might get trapped in the texture of the roughening, reducing the shear transfer capacity. It is recommended that a low viscosity (very pourable) grout with good “wetting” characteristics be selected.

6.4 Grouting Specific Connections

6.4.1 Precast Concrete Beam to Column Joint

Figure 6.9 illustrates a typical arrangement of a precast beam placed on a cast-in-place or precast column. Column bars pass through corrugated metal ducts cast in the precast beam. The choice of duct size is influenced by the likely size and location of the column bars passing up through the beam-column joint. The column bars, in an ideal situation, will be centrally located in the ducts. However, in reality, the column bars will vary from the

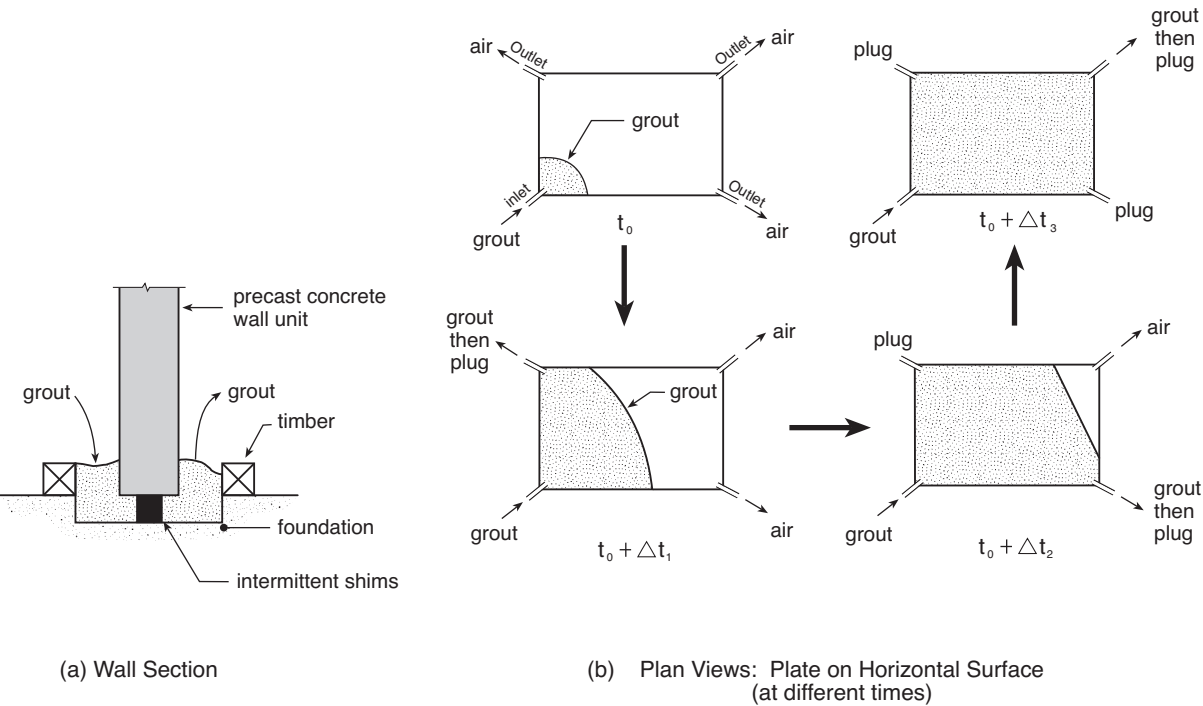


Figure 6.8: Horizontal voids or spaces

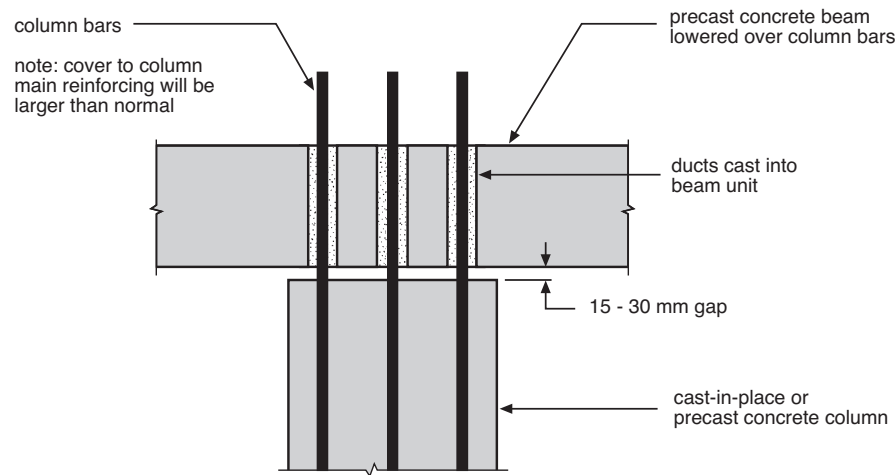


Figure 6.9: Schematic beam-column assembly

ideal positions, but should be within certain contractual or specified tolerances. A duct size should be chosen to accommodate these tolerances, plus a recommended additional 10 mm clearance to the bar to allow grout to flow between the duct wall and bar (Figure 6.10). Typically, duct diameters range from two to three times the nominal diameter of the grouted bar. (The Canadian Prestressed Concrete Institute [6.2] recommends three bar diameters for the internal duct size.)

Since column bars are ideally located at the centre of ducts, their cover will be larger than normal. This is because covers include the distance to stirrups or duct wall plus the clearance of the bar from the duct wall.

The two principal methods of grouting a precast concrete beam to column joint are as follows.

- **Method 1**

After sealing the gaps as for Section 6.3.3.2, grout (typically non-shrink cement-based) is pumped in

at one inlet port (or tube) to displace air progressively across the interface (Figure 6.8). If the grout has a relatively high viscosity, it may start to flow up the open ducts, starting at the duct closest to the grout inlet (Figure 6.11).

It is recommended that outlet ports be provided on the other three corners. These should be progressively plugged once grout without air bubbles flows out. Once the outlets are stopped, pumping of grout will continue to fill the ducts in the precast beam upwards from the bottom.

The duct nearest the inlet should fill first while the one furthest away (opposite corner) may require topping up by use of a tremie tube, or by pouring-in from a dispenser in such a way that grout runs down in contact with the reinforcing bar to avoid air-locks. The inlet tube or port is plugged once injection is completed.

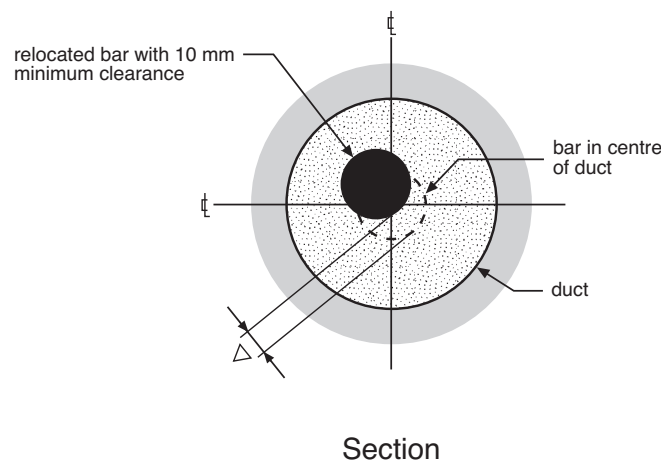


Figure 6.10: Column bar and duct clearance

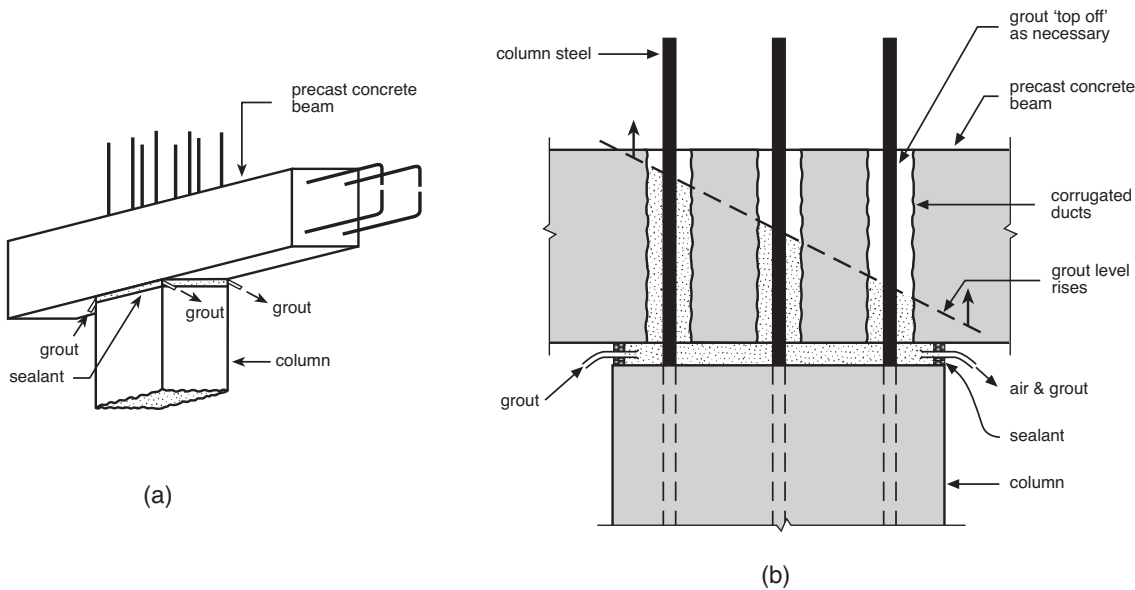


Figure 6.11: Grouting of a beam-column joint

- **Method 2**

The horizontal gaps are first sealed, then providing the ducts are sufficiently large, grout may be poured in from a dispenser down one corner duct. Progressively, the grout will flow across the column-beam soffit interface and up the remaining ducts. It is recommended that, as with Method 1, outlet ports be used to confirm the progress of the interface grouting. Topping off for ducts remote from the filling position may be necessary. Again, care must be exercised so that no air is trapped in the ducts.

In both Methods 1 and 2, outlet ports have been omitted. This produces a degree of uncertainty as to the extent of grout along the interface between the top of the column and the underside of the beam. Almost certainly there will be a small air-lock beside the duct at the furthest corner from the input duct or port. This practice is therefore not recommended.

Caution is required when sealing perimeter gaps. Sealing materials can reduce covers and may promote corrosion of reinforcement as discussed in Section 6.3.1.1.

6.4.2 Column Splices

Figure 6.12 shows typical grouted column splice configurations.

Some connections are made by bars protruding downwards from the precast element above, with the bars located in pre-grouted ducts or proprietary steel sleeves (Figure 6.12(b)(ii)). Care must be taken to ensure that:

- The sleeves or ducts, receiving bars from above, are clear of any debris or contaminants likely to reduce the bond or obstruct the placement of these bars.
- In the case of grout being placed in the receiving ducts prior to the bars being inserted, the bars must be placed before the grout has started to set. If this is not done, poor bond and partial bar penetration will result, leading to an inferior connection.

Other configurations consist of connecting bars which protrude upwards.

The precast units have ducts or proprietary sleeves to fit over the bars that extend upwards from the adjacent units. If required, the units may be correctly levelled on shims then grouted with a horizontal interface and vertical duct grouting, operating similarly to Method 1 (Section 6.4.1). Units may be seated on shims and a bed of cement or epoxy-based mortar consistent in strength and colour with the precast units. Shims should not obstruct the flow of grout and, if metal, the shims should have sufficient edge cover to prevent corrosion from occurring. If the mortar bed approach is used with the post-grout method, care is needed to ensure that the ducts and inlet ports are not blocked by intruding mortar.

Two types of ducts are typically used in column splices:

- proprietary steel sleeves; and
- corrugated metal ducts (Figure 6.13).

For grouting either type of duct configuration it is necessary to have an inlet tube at the base of each duct

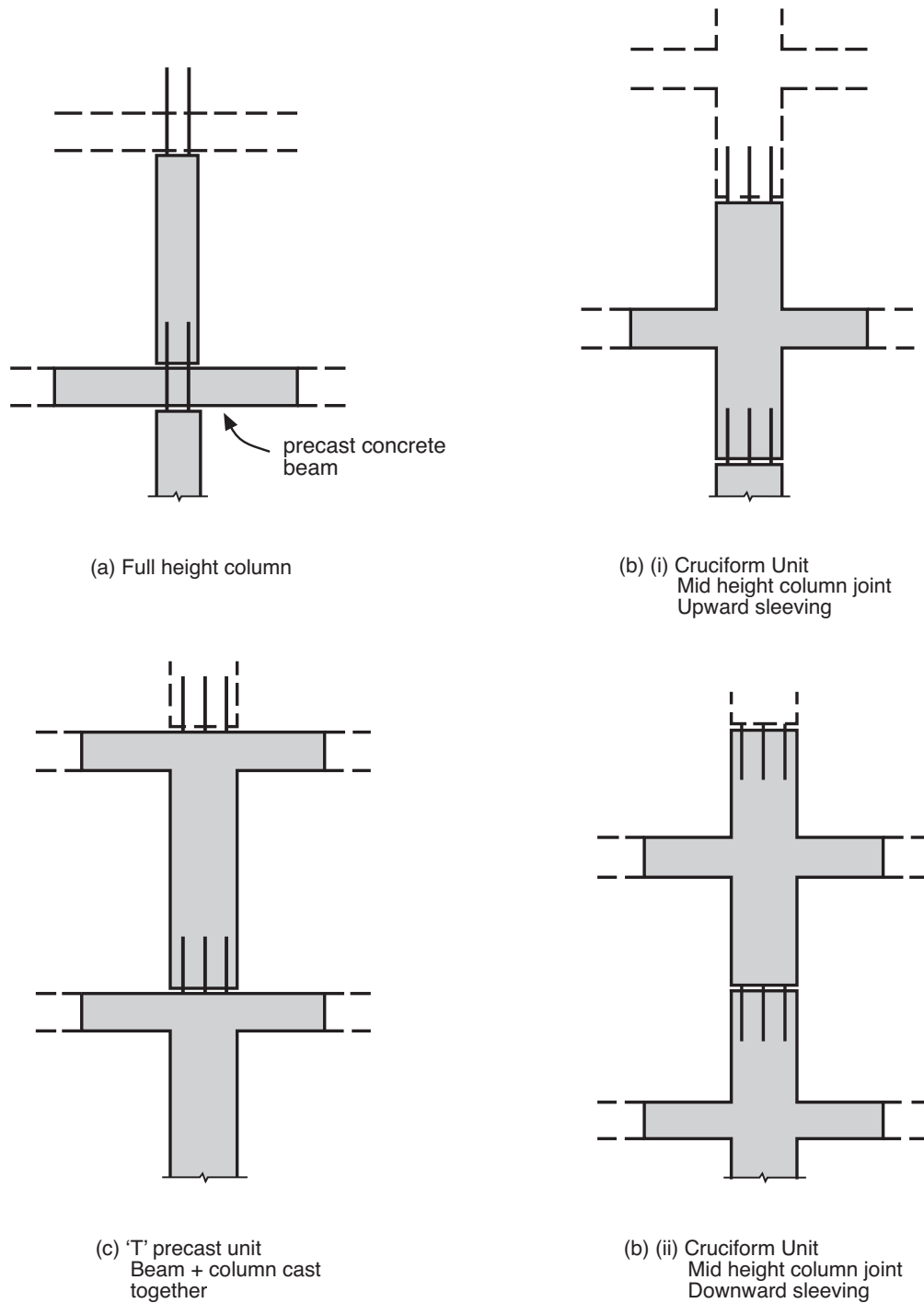


Figure 6.12: Typical column splice configurations

and an outlet tube at the top for bleeding air and grout (Figure 6.13). In practice, a significant amount of grout injection can be done from one interface inlet port or tube, and topping up as necessary for the individual ducts or sleeves through their lower inlet tubes. The grout may be injected under pressure that is maintained, as in grouting of post-tensioned cables in pre-stressed concrete construction.

A steel sleeve splice type system may require tighter control over tolerances both in precasting and site erection and a higher degree of supervision by the designer and construction staff may also be required.

Any proprietary splice system must be laboratory tested to show compliance with the criteria in the New Zealand Concrete Structures Standard [6.3]. For an

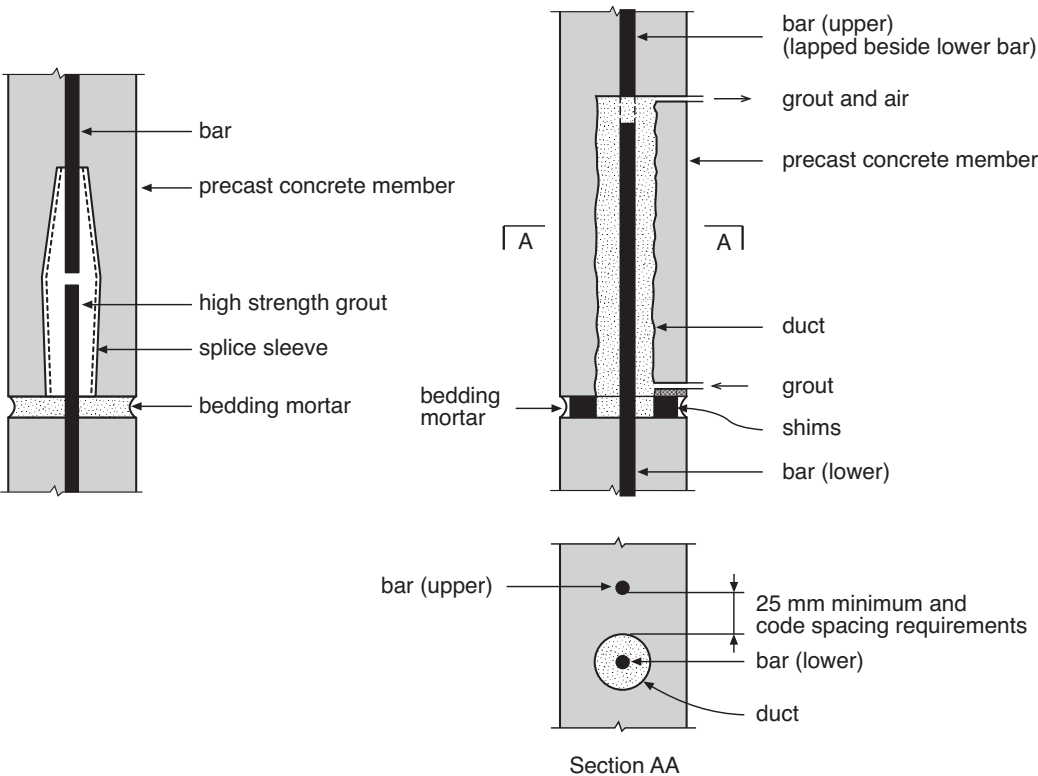


Figure 6.13: Steel sleeve splice and duct splice detail

example of a test report, readers are referred to Reference 6.4.

6.4.3 Beam Splices

Figure 6.14 shows the junction of two precast concrete beams linked by steel sleeve splices (other mechanical type couplers can be similarly installed). The installation procedure follows this general approach:

- Align the beam units so that the sleeve connectors (which are slid completely along the bars of one unit prior to alignment) can be slid over the bars of the second unit.
- Having slid the sleeves on to the bars of the second unit, ensure that the correct amount of each bar is occupying the internal space of the sleeve. Marks on the bars should be provided to assist with this.

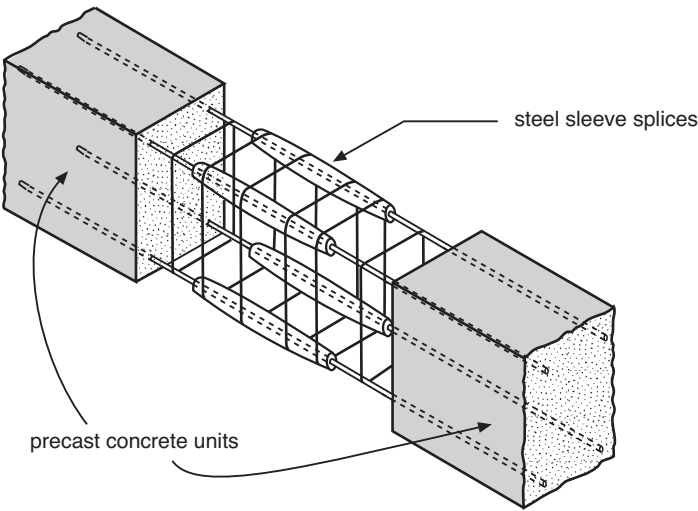


Figure 6.14: Proprietary reinforcing bar splices between precast concrete beams

- Seal sleeve ends and fill with the specified grout in accordance with the manufacturers'/specifiers' instructions. Ensure the sleeves are not disturbed until the grout has adequate strength or else the connection will be weakened.
- Place any other reinforcement and complete construction details to be cast-in in this zone.
- Place the required form-work for the final operation.
- Complete the connection of the two precast units by filling the beam form-work with cast-in-place concrete (the specified 28-day compressive strength of which should be at least equal to that of the precast elements).

Each proprietary coupler system will have its own particular method for installation, which should be strictly followed.

6.5 Quality Assurance

6.5.1 General

The designer should indicate the quality assurance requirements for any grouting operation. These requirements should be part of the contract documents, available at time of tender. If all parties are aware of quality assurance requirements at tender stage there should be fewer difficulties in implementing them during the contract.

Quality assurance is essential for the success of any grouting operation. Bar size and length, hole size, depth and cleanliness, grout type and quantity, pressurising procedures for the grout (if any), protection against disturbing the immature joint, and avoidance of airlocks all need careful attention and confirmation of the correct procedure.

In any detail where there are stages of partial completion, checks are appropriate. For example, when ducts are cast into units, checks should be made on type of ducts, location, length, etc. Similarly, for bars to be grouted-in, correct bar strength, size, embedment length and location within the duct should be checked.

At pre-definable stages the contractor should sign off each "milestone". This approach is currently being used in the precast industry quite successfully.

The designer, particularly during the initial stages, should co-operate with the contractor to ensure that the details are being correctly interpreted and built, and to monitor the quality assurance programme.

6.5.2 Assurance of Complete Grouting

From experience, it appears that the only practical method of establishing whether units are fully grouted is by grout volume measurement. The volume of the voids is calculated and this should equal the volume of grout that was inputted, minus the volume of grout collected from outlet or bleed ports. The degree of accuracy is subject to judgement and should be established prior to commencement of grouting.

6.6 Grouting Workmanship and Construction Aspects

6.6.1 Workmanship

The construction of grouted details and the grouting operation are specialised practices requiring high standards of workmanship. For example, details such as location of ducts to match others in adjacent precast units and the location and bar length requirements for grouted steel sleeve splices [6.5] require tighter tolerances than those for conventional reinforced concrete construction. Grouting should always be done by experienced applicators, either sub-contractors or trained personnel from the contractor's staff. It must not be considered as an operation to be done at some later, more convenient stage, by any available staff (typically lacking the necessary skills).

At tender stage, the contractor should be made aware of all grouting operations and any complexities so as to allow for the high standard of workmanship required.

6.6.2 Construction Aspects

Experience has shown that attention should be paid to the following matters during a grouting operation:

- Use of compressed air or a vacuum to remove dust from voids or spaces may not adequately remove dust adhering to the sides of the void, particularly if the concrete is of an early age or in a damp environment. Brushing with a stiff brush will normally be necessary in addition to the use of air or water for flushing out the void.
- If air is used to blow out dust and debris from ducts or voids, it should be ensured that either an oil filter has been installed, or no oil-bottle is or has been used on that line as part of any air-driven equipment. Any oil will damage the bond between the surfaces and grout.
- A major problem with grouting voids or gaps is the failure of sealing compounds or forms around the perimeter of the voids. Leakage through the seal-

ants not only complicates the measurement of grout volumes, but also causes unwanted delays. Surface finishes may also be damaged.

- Any excess grout which could reduce the bond to that surface should be cleaned off.
- Under no circumstances should bars be moved or knocked before the grout has gained sufficient strength. Such movement has a detrimental effect on the bond strength between the bars and grout.
- After cement-based grouting has been completed, grouted ducts should be protected from premature moisture loss.

6.6.3 Correct Usage of Grouts

The grout specifier and installer must ensure that the manufacturer's instructions for the product are followed exactly. If more detailed information is required, then the supplier/manufacturer should be approached for technical advice.

Concern has been expressed regarding the reliance placed on manufacturer's specifications. Depending on the integrity required for the grouted detail, the designer should be convinced that the manufacturer's details are appropriate. Further tests could be required from the manufacturer, or as part of the contract independent laboratory testing, and trial grouting of proposed details could be specified. Due allowances need to be included at time of tender.

6.6.4 On-site Testing

On-site pull-out tests may also be specified as a means of verifying correct workmanship and function of the connections. Where load tests are to be conducted on grouted-in bars, the test load may be limited to 85% of the tensile yield strength of the bar so as not to damage it for usage. This level of load is normally sufficient to identify improperly installed dowels. Usually any slippage or pull-out of the bar at this level indicates incorrect installation. The designer must, however, realise that there is still a degree of uncertainty that the grouted detail will reach its full design strength.

In order to verify the strength capacity of grouted bars, it may be necessary to pull embedded items to failure. "Failure" is either tensile rupture of the bar, concrete cone pull out, or bar slippage beyond a pre-specified limit. Such tests should be done on items that will not be part of the final structure.

In either the non-destructive method or the full failure method, the testing should not interfere with the general construction programme of the site.

6.7 Connections between Precast Elements using Welded Reinforcing Bars

6.7.1 General

The welding of reinforcement bars is a practical method of developing force transfer in many connections, provided that the weldability of the bars and adherence to sound welding practice can be assured. The three situations of welding reinforcing bars are as follows:

- To each other;
- Through splice members (such as angles, reinforcing bars, or flat plates);
- To structural steel members anchoring the bars.

Some of the common welds used with reinforcing bars are shown in Figure 6.16 and two typical examples of splicing reinforcing bars are shown in Figure 6.17. Figure 6.18 shows two welded steel details used to join precast concrete beams.

These guidelines have considered the welding of two new "seismic" grades of reinforcement, designated as "E" grade in the joint Australian/New Zealand Standard (currently being produced) for the production of reinforcing bars. Welding of bars that are not classed as "E" grade should be undertaken in accordance with requirements of the joint Australian/New Zealand Welding Standard AS/NZS 1554.3.

6.7.2 Welded Reinforcement Splices

A new joint Australian/New Zealand Standard for the production of reinforcing bars, which is to supersede NZS 3402 [6.7], lists two grades of reinforcing bar that meet seismic requirements, and hold the "E" (for Earthquake) designation mark. These two seismic grades 300E and 500E are high ductility grades in the 300 MPa and 500 MPa lower characteristic yield strength category.

The New Zealand Concrete Structures Standard [6.3] defines the requirements of welded splices and mechanical connections when used in given structural locations. The designer must ensure that weld details being proposed comply with this code and to any additional requirements of the joint Australian/New Zealand Welding Standard AS/NZS 1554.3 [6.9], which is replacing NZS 4702 [6.6]. At the time of writing this guideline, AS/NZS 1554.3 was still in the draft stage and the current NZ Standard was NZS 4702. However it is noted that NZS 4702 does not cover the new New Zealand produced micro alloyed seismic grades 300E and 500E.

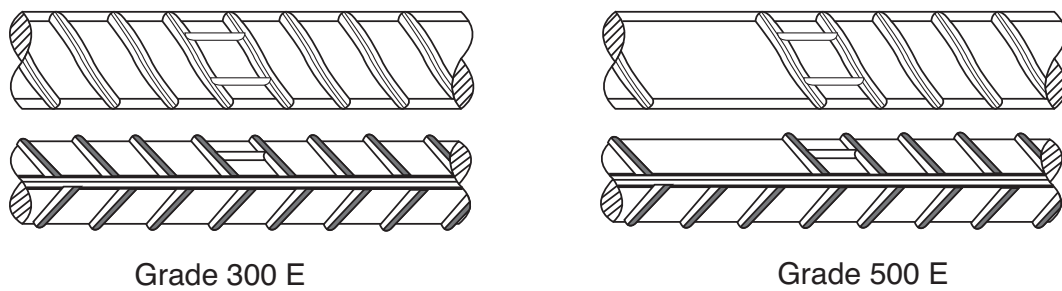


Figure 6.15 : Identification Marking of Grade 300E and 500E

Parameters controlling ductility such as minimum and maximum yield strength, tensile to yield strength ratio and uniform elongation are set tighter than for the low ductility (L) and normal (N) ductility reinforcement bars. When joining these bars by welding, these properties need to be maintained ensuring that the weld has sufficient tensile strength for it not to be the weakest link. However the weld should also contribute to the deformation requirements and therefore the weld should provide yielding close to the yield of the parent bar combined with acceptable elongation.

The current Grade 300E and Grade 500E manufactured in New Zealand are of the micro alloyed type produced by hot rolling. The “micro alloyed” method uses small, closely controlled additions of alloying elements such as vanadium, which increase strength, overall ductility and assist in maintaining good weldability. All grades manufactured to the anticipated joint Australian/New Zealand standard for the production of reinforcing will carry specific mark-ups to identify the bars and to recognise the specific welding requirements. Figure 6.15 shows the marking to the new reinforcement production standard for the 300E and 500E reinforcement bars.

An extensive study by NZ HERA [6.11, 6.12] on the weldability of the micro alloyed Grades 300E and 500E produced by Pacific Steel proved their excellent weldability and mechanical properties, provided the correct welding procedures are followed. The study also showed that for seismic applications the appropriate choice of joint type is of importance.

6.7.3 Selecting Joint Types for Seismic Applications

Using “seismic” grade reinforcing bars, the correct welding consumable, and following qualified welding procedures guarantees that well executed welded joints will satisfy the mechanical strength and ductility requirements, for seismic loads in the plastic range within the design expectation. This is provided the joints are symmetric to the central axis of the applied

loads. Therefore butt joint configurations or bar to plate welds as shown in Figure 6.16 will perform well if loaded past the yield point in the inelastic range.

However, non-symmetric joints such as the standard lap joints will develop a moment of rotation, which can lead to brittle fracture at the weld ends if stressed past yield. This effect particularly shows at the higher strength 500E bar and in larger diameter. The 300E Grade performed with satisfactory ductility in the tests up to and including the 32 mm bar diameter. If splice joints are jointed by indirect means like using angle backing or flat backing (see Figure 6.16), the dimensions used for the angle or the backing plate will determine the degree of bar rotation and if premature fracture will occur. Therefore careful consideration for plate and angle dimensions needs to be given. Alternatively, the indirect butt splice joint alternative (two bar pieces in a symmetrical lap splice) as shown in Figure 6.16 can be chosen. Due to its symmetric nature ductility performance is excellent.

Extending the overlapping length of the lap joint is also an option and provided the overlapped length is sufficiently long the resulting moment (resulting from the eccentricity of the bars) is sufficiently low to not sustain any brittle fracture even in the heavier thickness higher strength bars.

Similar to the non-symmetric lap joints, the precast concrete detail with the bent and welded reinforcing bar ends (Figure 6.18) should be avoided for seismic applications especially in the higher strength Grade 500E and larger diameter bar alternatives. Symmetric alternatives such as shown in Figure 6.18(a) are preferred to avoid brittle fracture if stressed past yield.

When specifying a joint detail, the designer must be aware that for some joints it is difficult for the welder to readily achieve the required weld quality. Joints that provide for good access of the welding arc to the root of the weld are preferred. For example, the bar to plate joint configuration shown in Figure 6.16, the single side fillet weld (detail on the left) or the double side

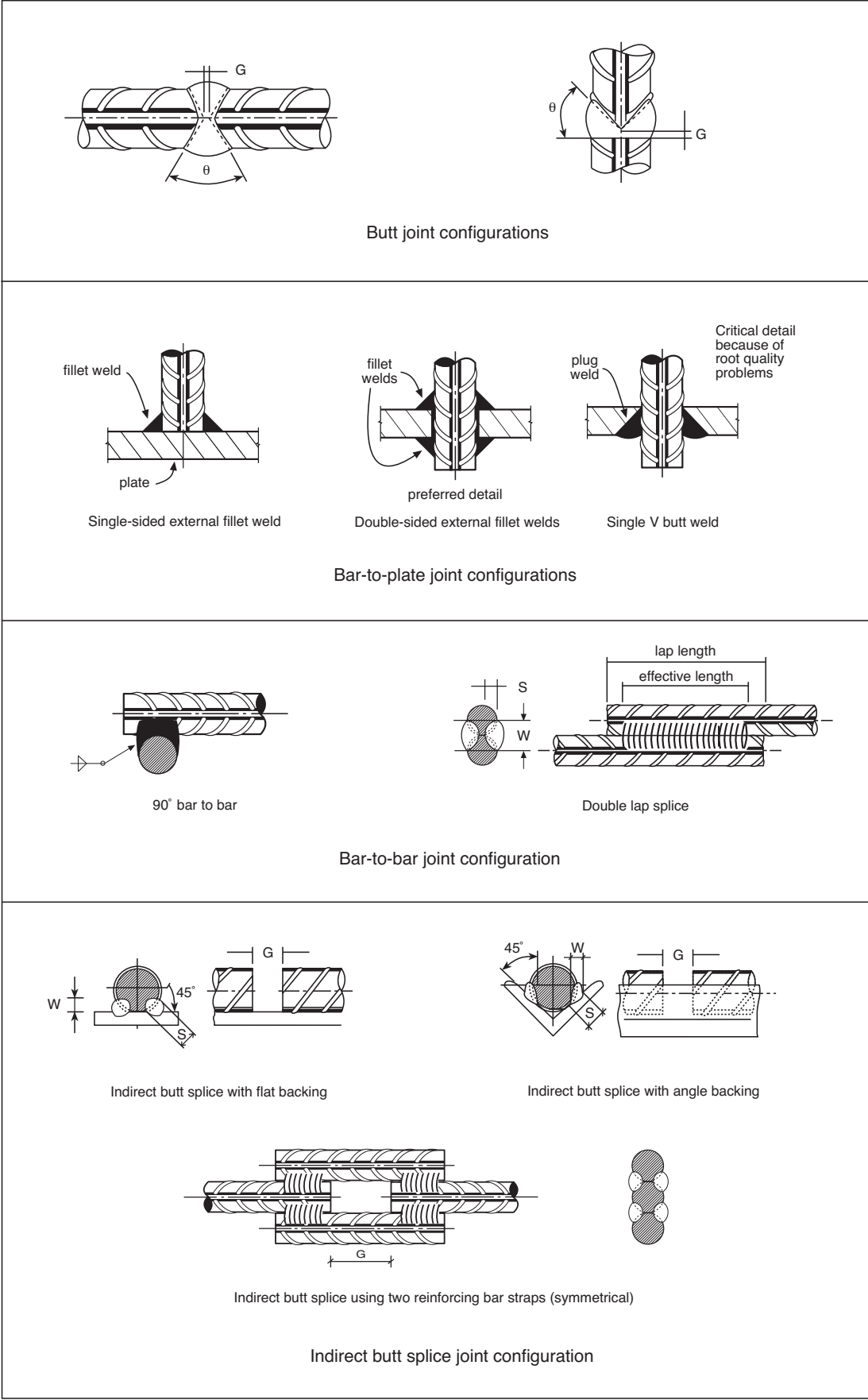


Figure 6.16 : Typical reinforcing bar welds

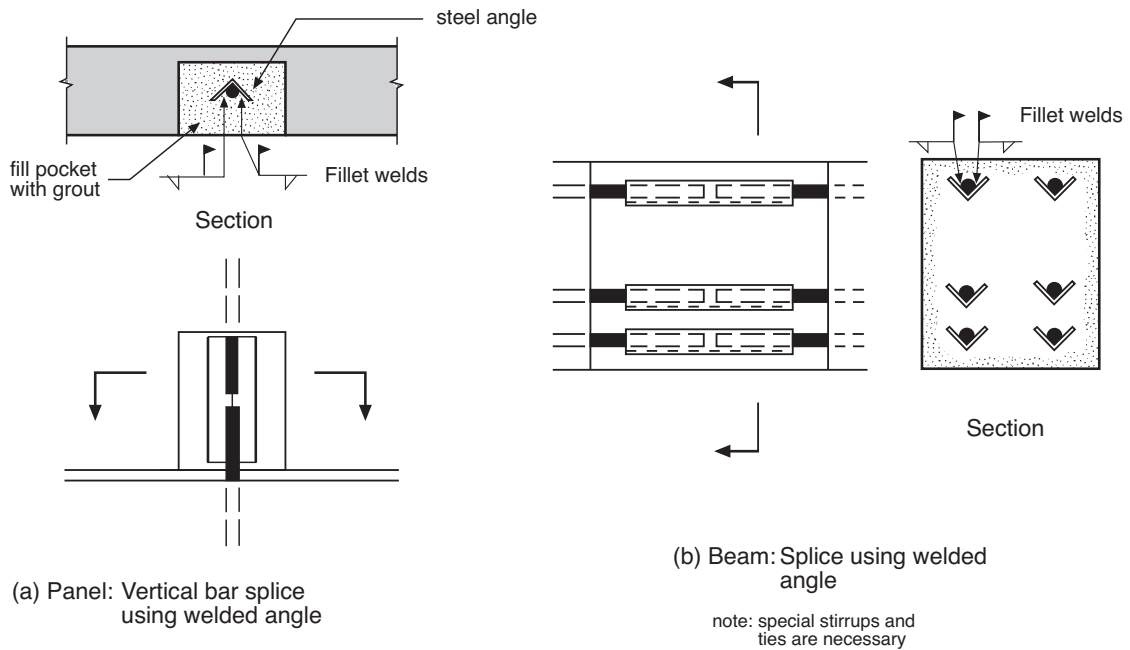


Figure 6.17 : Examples of splicing reinforcing bars

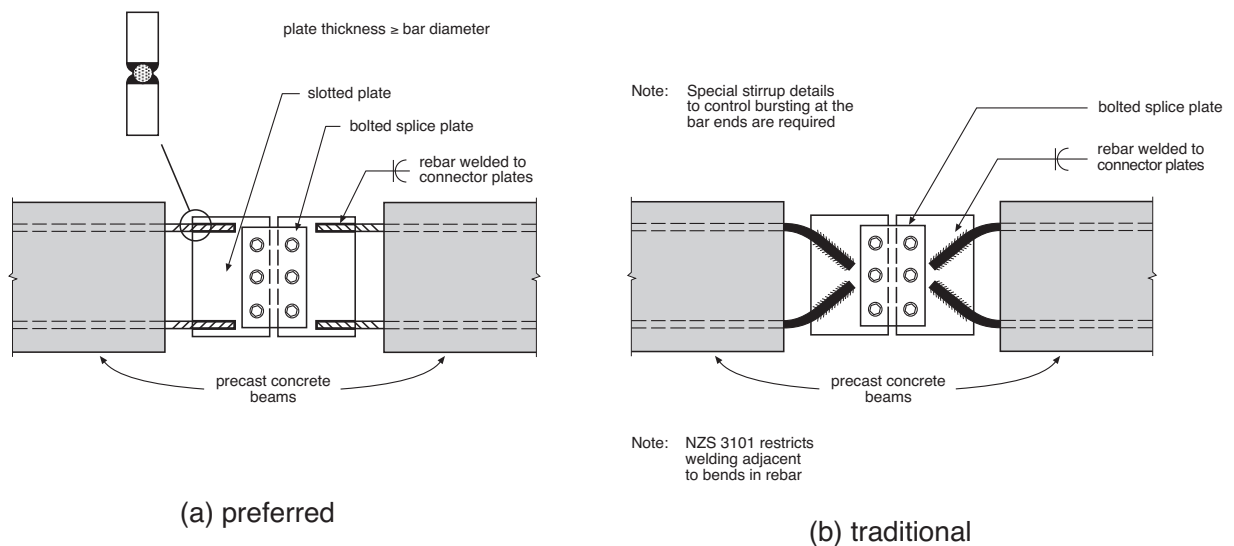


Figure 6.18: Bolted splice connection between precast concrete beams

fillet weld (detail in the centre), is much preferred to the single V butt weld (detail on the right). In the newly revised AS/NZS 1554.3 [6.9], this “prequalified” joint alternative considers the difficulty of weld access by requiring an additional 2 mm of weld throat, plus consideration of the maximum possible gap between the bar and the corresponding plate hole. In the single V-butt weld detail considerable care is needed to obtain a satisfactory weld in the root area of the joint due to the poor access for the welding arc. Verification

of the cross sectional area of the weld achieved can also be costly because weld inspection is difficult. For the fillet weld alternatives, not only is access for the welder easily achieved, the weld quality can be readily confirmed by visual inspection.

The 90° bar-to-bar weld detail as shown on Figure 6.16 is not a prequalified joint and, like the double lap splice, it should not be chosen if full plastic performance is required. Welding procedures for the 90° bar to bar detail need to be fully qualified by the contractor.

6.7.4 Weld Specifications

As previously stated, welding of reinforcing bars in New Zealand is covered in AS/NZS 1554.3 [6.9]. The principle of this standard is that satisfactory weld quality is achieved by the use of qualified materials (bars and consumables), using qualified welders following qualified procedures under adequate supervision. Welding inspection as defined in the standard supports this.

While the standard should be consulted on all aspects of welding, some general comments of interest to the specifier follow:

Preheating

Generally no preheat requirement applies for the Grade 300E and 500E manufactured in New Zealand by Pacific Steel. Exceptions may be called for in AS/NZS 1554.3 [6.9] for specific joints of heavier thickness bars (≥ 32 mm) where the joints configuration combined with low energy input may lead to fast cooling and preheat may be necessary. However, it is good practice, especially in cold and damp conditions, to consider slight preheat (30°C) to dry off any moisture around the weld site, reducing the risk for hydrogen induced cracking, lack of fusion and weld porosity.

Welding Process

The choice of the welding process should be left to the fabricator. Manual Metal Arc Welding (MMAW), Gas Metal Arc Welding (GMAW), Flux Cord Arc Welding (FCAW) gas shielded and FCAW self shielded are suitable. Well suited to on site welding are the MMAW and the self shielded FCAW process. In the workshop the GMAW and the gas shielded FCAW process alternatives are best suited and more economic. While for the MMAW and FCAW of the 300 Grade non-low-hydrogen consumables are acceptable, for the 500E grade only low hydrogen consumables are recommended.

Welding Procedure Specification

AS/NZS 1554.3 [6.9] requires the contractor to establish welding procedures. These should be made available to the welder and on request to the principle's representative who is usually the inspector or the engineer. The welding procedure details all essential variables including joint preparation and welding settings which determine repeatable weld quality. As the welding procedure qualification is based on testing, it has been verified that these parameters produce a satisfactory joint by an adequately experienced and qualified welder.

Qualification of Welding Personnel

AS/NZS 1554.3 [6.9] requires that a suitably qualified welder under the supervision of a suitably qualified welding supervisor shall carry out all welding. It is important to note that the welder needs to be qualified in both the welding process and for the welding position used. Welder qualification can be specific to the on the particular procedure applied, or in the form of wider qualification such as NZS 4711 [6.10].

Inspection

AS/NZS 1554.3 requires that all welding shall be subject to the examination of an inspector appointed by the Principal. This inspector shall be suitably qualified and shall inspect the work prior and after welding to the requirements of the standard.

Welding near bends in reinforcing bars

An example of where this welding detail occurs can be seen in Figure 6.18(b). NZS 3101 [6.3] states that welding can not be undertaken any closer to a bend (including that of a restrengthened bar) than three times the diameter of the bar being welded.

6.7.5 Tolerances

Particular care is necessary when fabricating precast concrete units joined by welding. Bars to be joined by butt welds require accurate alignment. Welded splices with steel angles in most cases require less accuracy. The designer must be aware of the achievable accuracy of alignment for welded details and check that this does not exceed the maximum permitted by NS/NZS 1554.3.

6.7.6 Welding Information for Grade 300E and Grade 500E Reinforcement Bar Manufactured by Pacific Steel

Should any queries outside AS/NZS 1554.3 [6.9] arise, it is recommended that they be directed to HERA's New Zealand Welding Centre. A large number of welding procedures covering MMAW and GMAW in the most common joint alternatives and welding positions have been developed and qualified, and copies of the procedures can be made available to product users.

6.8 Mechanical Connectors for Splicing Reinforcing Bars

The performance of mechanical connectors must comply with the New Zealand Concrete Structures Standard [6.3]. Various forms of these connectors are available in New Zealand. The principal method used is butt splicing.

Types of mechanical connectors are as follows.

- **Threaded bars**

- Ends of bars are threaded to accept a threaded sleeve coupler.
- Threaded sleeves that are swaged onto the ends of bars and a threaded coupler used to join them.
- Some bar types have the deformations for bond roled as threads, where proprietary sleeves with matching internal thread to those deformations are used to make the splice.

- **Cadweld**

A proprietary system consisting of a sleeve into which a molten metallic filler is poured. Once the filler has solidified, the splice is complete.

- **Swaged sleeves**

The splice is achieved by the swaging of a proprietary sleeve on to the ends of butted bars.

In all cases these splicing systems, as with butt welds, require a high degree of accuracy for alignment of bars. These systems are perhaps most effectively used for a connection where there is at least one “free” bar which can be adjusted on site.

Most proprietary systems offer a range of strengths for their couplers. Some couplers are designed to reach 100% or 125% of the yield strength of the bars. The designer must ensure that the system nominated meets the design and code requirements.

As with all proprietary systems, the manufacturer’s instructions for usage should be fully adhered to.

6.9 Recommendations

6.9.1 Connections made by Grouting of Precast Concrete Components

It is recommended that:

- When cement-based grouts are used they should be high strength and shrinkage compensating. The minimum grout compressive strength should be 10 MPa greater than that of the surrounding concrete. (Section 6.2.2.2.)
- For all grouting situations, extreme thoroughness with respect to cleanliness and the following of manufacturer’s instructions is required. (Section 6.3.1.)
- When bars are grouted in horizontal or inclined

holes, bar locaters should be used to keep the bars in the centre of the holes. (Section 6.3.3.1.)

- When precast concrete beam-to-column joints are being grouted, one of the two methods in Section 6.4.1 should be used, together with a low viscosity grout with good “wetting” characteristics.
- Designers should communicate to contractors at tender stage all the specialised requirements for the grouting operations, including the need for experienced operators and a satisfactory quality assurance programme. (Sections 6.5 and 6.6.)
- The grout volume method should be used to determine whether or not the units have been fully grouted. (Section 6.5.2.)
- Before compressed air is used to blow out dust from holes, it should be tested for oil contamination. (Section 6.6.2.)

6.9.2 Connections made by Mechanical Splices

It is recommended that when using mechanical splices designers should ensure the strength of the splice meets design and code requirements, and ensure that the manufacturer’s instructions for using the splices are followed in full.

6.10 References

- 6.1 Thurston, S J. *Precast Grouted Beam-Column Joint*, Report 5-86/9, Ministry of Works and Development, Central Laboratories, Gracefield, September 1986.
- 6.2 *Metric Design Manual: Precast and Prestressed Concrete*, 2nd Ed., Canadian Prestressed Concrete Institute, 1987, Canada.
- 6.3 *Concrete Structures Standard Part 1 and 2: NZS 3101:1995*, Standards New Zealand, Wellington, 1995.
- 6.4 Yong, P M F. *The Performance of NMB Splices for Grade 380 Bars under Cyclic Loading*, Report 5-85/9, Ministry of Works and Development, Central Laboratories, Gracefield, 1985, 60 pp.
- 6.5 *NMB Splice Sleeve System User’s Manual*, NISSCO Splice Sleeve Japan, Ltd.
- 6.6 *Metal Arc Welding of Grade 275 Reinforcing Bar, NZS 4702: 1982*, Standards Association of New Zealand, Wellington, 1982, 23 pp.

- 6.7 *Steel Bars for the Reinforcement of Concrete, NZS 3402:1989*, Standards Association of New Zealand, Wellington, 1989, 16 pp.
- 6.8 *General Information Sheet: G001: Reinforcing Steel Bar Welding Procedure*, Pacific Steel Ltd, Otahuhu, New Zealand, 4 pp.
- 6.9 *Structural Steel Welding - Welding of Reinforcing Steel, AS/NZS 1554.3 (under revision)*, Joint Standard, Standards Australia and Standards New Zealand.
- 6.10 *Qualification Tests for Metal-arc Welders, NZS4711:1984*, Standards Association of New Zealand, Wellington, 1984, 31 pp.
- 6.11 Scholz, W and Roberts, B. "Welding newly developed, high strength, seismic grade reinforcing bars", *12th World Conference on Earthquake Engineering*, Auckland, New Zealand, February 2000.
- 6.12 Scholz, W et al. *Welding Pacific Steel Reinforcement Bars Phase 1 + 2*, New Zealand Welding Centre Tech. Rep. TR9807, HERA, 1998, 1999.

Chapter 7

Embedded Steel Connectors

7.1 Introduction

Embedded steel connectors are used extensively by the precast concrete industry. Although not used for connections between primary precast concrete seismic resisting members, embedded steel connectors may be subject to significant gravity and earthquake induced forces resulting from:

- inertia effects, i.e. parts and portions provision of the New Zealand Loadings Code NZS 4203:1992 [7.1]; and
- imposed deformations, as secondary structural (Group 2) elements, under seismic loading, as required by the New Zealand Concrete Structures Standard, NZS 3101:1995 C1. 4.3.5, 4.4.13 and 4.4.14 [7.2].

Steel embedments may also be required to accommodate considerable localised movements resulting from concrete volume changes, member elongation under seismic loading, or temperature-induced structural deformations in fires.

In many cases the applied force or deformation calculated is at best an estimate, and the design of steel embedments should always be carried out conservatively, bearing in mind the consequences of failure. The deformation capacity of embedded connectors is therefore important, as the dependable strength may be reached and deformations may well exceed those calculated.

This chapter gives information on, and recommendations for, the design and detailing of steel connectors embedded in concrete, and interprets relevant provisions of the New Zealand Concrete Structures Standard.

7.2 Types of Connectors

The types of embedded connectors considered are shown in Figure 7.1, (a) through (e). These include:

- (a) and (b) weld plates;
- (c) cast-in steel sections;
- (d) post-drilled fixings, i.e. resin or expansion type; and

(e) cast-in concrete inserts.

A comparison of the advantages and disadvantages of each type of fixing is set out in Table 7.1.

7.2.1 Weld Plates

This type of connection (see Figure 7.1(a) and (b)) is cast into concrete members to allow for later attachment of similarly-equipped concrete members, or a steel member, by welding to the exposed steel surface.

7.2.2 Cast-in Steel Sections

Channel, angle, flat, hollow or universal sections may be cast into precast concrete to provide a joint to another member (Figure 7.1(c)).

7.2.3 Post-fixing to Precast Concrete

Connections can be formed by drilling into hardened concrete and inserting expanding anchors or resin-type anchor studs (see Figure 7.1(d)). Expanding anchors rely on friction or mechanical interlock (particularly where using undercut installation) to transfer axial tension and may need to incorporate low strength reduction factors (high factors of safety) to cover performance sensitivity to installation uncertainties. Note that in situations where the vertical gap between a precast concrete unit and connecting member is to be filled with cast-in-place concrete or grout, a preferable detail is to make use of an inverted steel angle. This avoids prying of the embedded fixing.

7.2.4 Cast-in Steel Inserts

Precast members may utilise cast-in steel inserts, which subsequently receive a bolt or threaded bar for connecting other structural elements (see Figure 7.1(e)).

7.3 Selection of Fixings

Before detailed design commences, a careful choice of fixing type should be made. There are a large number of factors to be considered. The checklist provided in Appendix D1 may be used when making a selection for a given situation. Other sources of information on mechanical fasteners in concrete are given in References 7.3 and 7.4.

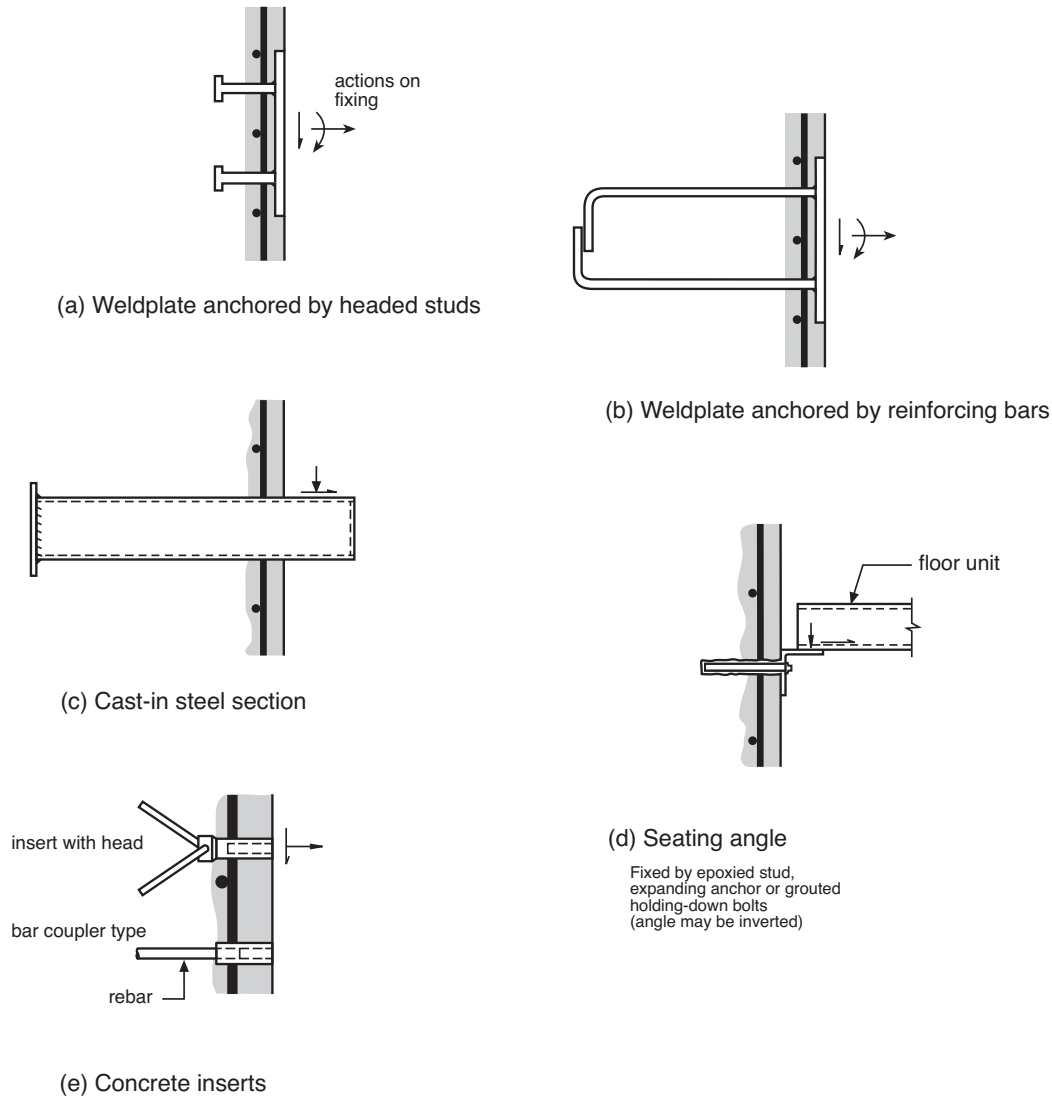


Figure 7.1: Types of steel embedments

Specific questions requiring consideration are :

- What loads are to be carried by the fixing assembly?
- What deformation is the fixing assembly required to withstand?
- What load/deformation characteristic is required under overload conditions?
- What kind of fixings are available for this application?
- What are their performance characteristics?
- Are these all the factors that influence the performance of the fixing assembly?
- What about corrosion?
- What has to be considered in the structural design of the fixing assembly?
- How sensitive is the fixing assembly to a shift in the position of applied load, or dimensional changes, etc?
- Are there any potential problems with the method of assembly, or installation?
- How does the designer convey performance requirements to the user?
- Is any load verification testing required?
- Are there any other matters to be considered?

7.4 Structural Actions on Steel Embedments

7.4.1 Loads

The most common load types that may act on steel

Anchor Type	Advantages	Disadvantages	Overload Characteristics
Weld Plates Figure 7.1(a) 7.1(b)	Reliable installation Reliable strength prediction No load test required	Positioning accuracy (small tolerances)	Reliable when anchorage design based on ductile yield in steel
Cast-in steel sections Figure 7.1(c)	Reliable installation Reliable strength prediction	Difficulty of concrete compaction around steel Positioning accuracy (small tolerances)	Reliable when anchorage design based on ductile yield in steel
Post-fixed anchor Epoxy type Figure 7.1(d) Undercut anchors	Positioning accuracy easily achieved Reliable strength prediction	Load performance sensitive to installation method Creep effects impose limitation on magnitude of long term loadings Verification load tests required See cast-in inserts	Reliable when anchorage design based on ductile yield in steel and verified by load tests
Post-fixed anchor Expanding type Figure 7.1(d)	Positioning accuracy easily achieved Proprietary components Ease of removal	Load performance sensitive to installation method Proprietary types have limited embedment depth, i.e. difficult to anchor into confined concrete	Overload strength limited by concrete tension/anchor slip Ductility achievable by using specialist components, e.g. bar splicers
Cast-in inserts Figure 7.1(e)	Reliable installation Reliable strength prediction Proprietary components	Positioning accuracy (small tolerances) Proprietary types have limited embedment depth, i.e. difficult to anchor into confined concrete	Overload strength limited by concrete tension/anchor slip Ductility achievable by using specialist components, e.g. bar splicers

Table 7.1: Comparative advantages of types of embedded connectors

embedments are identified, together with appropriate combinations in the loadings code [7.1].

7.4.2 Actions Resulting from Movements of Structural Elements

As discussed in Chapters 2 and 5, buildings incorporating precast concrete elements are subject to concrete volume changes due to temperature effects, shrinkage and creep. In buildings where lateral seismic loads are resisted by ductile seismic frames, movements of far greater magnitude may occur due to the elongation of beam plastic hinges. This phenomenon is described in Appendix A.1.

In most cases steel embedments do not have sufficient strength to prevent such movements from occurring, so the design and detailing must provide deformation capacity through either flexibility or ductility. It is

therefore important that joint axial deformations and rotations be correctly assessed, with some reserve to allow for inaccuracies in the global analysis. The change in position of reaction points as deformation occurs needs to be assessed. Friction loads should be considered when an element moves in response to volume changes, as illustrated in Figure 7.2.

7.5 Design Approach

The strength of a steel embedment is governed by either steel strength, the concrete resisting mechanism in epoxy type post-tensioned anchors, or also by bond failure. The concrete resisting mechanism is itself dependent on a number of factors. These include concrete strength, embedment length, edge distances, anchor spacings and other less frequently considered factors such as the presence of cracks, potential loss of

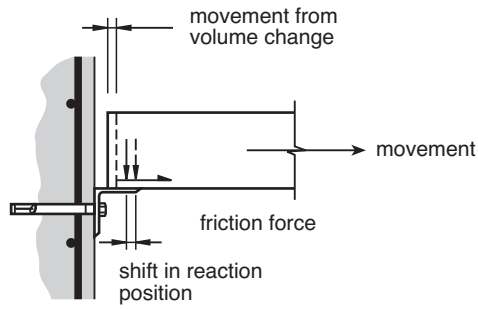


Figure 7.2: Friction load acting on a steel embedment

concrete cover and the type of loading (monotonic or cyclic) and the method of installation.

In an overload situation the steel component of the embedment is generally ductile whereas the concrete resisting mechanism is brittle. Brittle failures in the steel component may still occur in bolts or in short connections, where the ductility of the connector over a small length results in a very limited deformation capacity. Also, brittle failures may arise in bolted connectors subjected to cyclic loads in shear or combined tension and shear. Capacity design, where the strength of the steel embedment is required to be less than the dependable strength of the concrete resisting mechanism, is the recommended design approach. This approach should be used not only for seismic loads, but also for any other type or combination of loads, so that in all situations brittle failure of the concrete resisting mechanism is prevented. Capacity design is particularly relevant when the effects of concrete movements are considered since, as previously mentioned, the forces that are generated may reach the strength of the connection.

7.6 Design Methods

7.6.1 General

Many methods have been proposed for design with embedded steel connectors. Designers should be cautious when choosing a design method, since the limitations of the method may not always be stated, or the equations may have been calibrated to obtain mean values rather than the 5% lower characteristic strengths. Some methods underestimate the detrimental effects on the concrete resisting mechanism caused by edges in the anchoring element or by overlapping of the potential surface failures. A recent state-of-the-art report [7.4] highlights the limitations of many design methods available.

The design philosophy in the Concrete Structures Standard [7.2], Cl. 4.3.5(a) and (c) is rational. The

principle objective of Cl. 4.3.5(c) is to demonstrate that yield or slip occurs in connectors in order to preclude a brittle concrete pull-out failure. This implies that, while the connector is sized for the design loads, the embedment length in the concrete is proportioned to ensure the development of yielding in tension in the connector itself, particularly in statically indeterminate structures or components.

Note that not all proprietary embedments, which quote ultimate limit state loadings, satisfy the above criteria and these cannot therefore be “prequalified” for use in seismic resistant applications.

To calculate the embedment length, it is important to incorporate the statistical variation inherent in the factors contributing to the capacity, or deformation demand, in the connector.

The main parameters affecting these are:

- the actual tensile or yielding strength of the connector surrounding the embedment;
- accuracy in establishing the embedment length of the connector;
- scatter between observed and predicted capacity — design equations are based on “best-fitting” of experimental results over a range of values;
- long-term performance of the embedment, given its reliance on friction forces (which may decrease as a result of creep or cracking in the surrounding concrete) or sensitivity of its capacity to installation procedures;
- likely overstrength that can develop in steel connectors, where extensive yield strains are likely, and the effects of high material strengths and strain hardening contribute to the overstrength design actions applied; and
- consequences of failure of the embedment.

The consequences of failure should be accounted for in the design of an embedment. In lieu of a particular study, the component safety index should be $\beta = 3.5$, a value considered appropriate for a near brittle pull-out failure. This concept is illustrated in Figure 7.3. In Reference 7.5, the second moment probabilistic methods are used to derive such factors. The variation in material strengths, embedment length and curing of the concrete were considered in the study. With reference to Figure 7.3, it was proposed that $\lambda = 1.21$ and $\phi_c = 0.51$ be used to modify the 5% lower characteristic strength of the connector and the mean value predicted for the concrete resisting mechanism respectively (λ = material factor for steel and ϕ_c = strength reduction factor

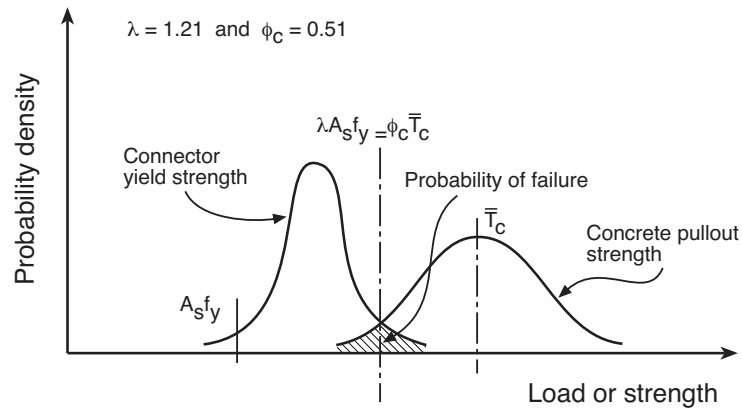


Figure 7.3: Derivation of the load factors for embedments

associated with the concrete cone pull-out failure). The design procedure and factors proposed in Section 7.6.2 are consistent with the values proposed in Reference 7.5.

7.6.2 Weld Plates with Headed Studs

It is recommended that the design of a connection using weld plates with headed studs be carried out in two stages. In the first stage the dimensions of the studs are determined using a strength reduction factor for tension, shear or combined tension and shear equal to $\phi = 0.85$ when capacity design is not used, and $\phi = 1$ when the actions have been derived using capacity design. In the second stage the embedment length of the studs or group of studs is then determined using capacity design to preclude a brittle failure of the concrete resisting mechanism. In this second stage, it is explicitly assumed that a connector with a well-defined yield plateau will develop the yield strength in tension, and that connectors without a yield plateau will develop the ultimate tensile strength. The dependable strength of the concrete resisting mechanism should be based on equations derived to predict the 5% lower characteristic strength multiplied by a strength reduction factor equal to $\phi = 0.85$ when capacity design is not used and with $\phi = 1$ when the actions have been derived using capacity design.

The recommended design procedure generally follows that outlined in Reference 7.4 with the following proposed modifications:

- the procedure for designing steel embedments in light-weight concrete should be modified to account for the brittleness behaviour of this concrete type;
- effects of spalling of cover concrete should be allowed for, in accordance with Section 7.6.7; and
- the cover for ductility and fire resistance shall be in accordance with Sections 5 and 6 of Reference 7.2, respectively.

The recommended design steps, incorporating the above modifications, are as follows:

- establish weld plate and stud geometry, and the material properties;
- proportion the studs to carry the design actions;
- calculate the embedment length of the studs:
 - where shear load is applied towards a free edge, check the shear edge distance or provide hairpin reinforcement to carry the entire shear force;
 - check minimum tensile edge distance against actual edge distance available;
- check service load stresses in studs and in the concrete and, where cyclic loading is applied, a fatigue check is also required; and
- check effects of other factors, e.g. spalling of cover concrete on load resistance.

For cyclic loading, laboratory cyclic shear load tests [7.4, 7.6] indicate that, provided the stud embedment is adequate, the steel will fail due to the effects of low cycle fatigue.

Under cyclic loads such as might occur on an embedment supporting lift machinery, the design stress should be reduced as recommended in Reference 7.7 to allow for fatigue.

Two worked examples of embedment design are presented in Appendix E1.

7.6.3 Weld Plates Anchored by Hooked Reinforcing Bars

Weld plates anchored by hooked reinforcing bars present similar design considerations to weld plates with headed studs and design can follow the procedure

described in Section 7.6.2. The embedment length of hooked bars is slightly larger than for studs [7.5]. In elements where a reinforcing steel cage is present and a concrete pull-out failure is prevented from occurring due to the presence of well-anchored transverse reinforcement, the embedment length of hooked reinforcing bars can be obtained from the requirements of the New Zealand Concrete Structures Standard [7.2].

7.6.4 Concrete Inserts and Hooked Starter Reinforcing Bars

Headed steel inserts and hooked starter reinforcing bars are similar to headed studs and design can follow Section 7.6.2.

The load resistance of shallow concrete inserts or hooked starter reinforcing bars can be adversely affected by spalling of cover concrete. In situations where loss of cover concrete may occur, the inserts or hooked starter reinforcing bars should not rely on the tensile strength of and bearing on cover concrete. Transfer of the design load to the member should then be achieved by use of an additional U-shaped reinforcing bar passing through the eye-hole in the base of the insert and anchored in the core of the section, or the use of threaded bar couplers also anchored within the core (see Figure 7.1 (e)).

The use of inserts without bars through the eye-holes is not recommended, unless the embedment length of the insert is such that a brittle failure of the concrete resisting mechanism is prevented. Note that some proprietary inserts include eye-holes whose diameter is less than that required to accommodate the bar size necessary to resist the ultimate pullout load on the insert. These inserts should not be used when subject to seismic loading. The effect of adverse influences discussed in Sections 7.6.7 and 7.6.8 should always be considered.

7.6.5 Post-drilled Fixings

Post-drilled fixings, involving the installation of the anchor with epoxy, or non-shrink cement grout, can be designed using References 7.4 or 7.8. Post-drilled fixings, involving the use of expansion anchors, can be designed using a similar approach to that shown in Reference 7.4.

In general, the effect of concrete cracking on expansion anchors is significant (see Section 7.6.7) and complete loss of preload can occur if cracking coincides with the anchor location.

Proprietary post-drilled fixings, exhibiting the following features, are known to provide improved load resistance over conventional expanding anchor types:

- ability to pre-load the anchor against a load indicat-

ing washer to a level that ensures the externally applied load never exceeds the long-term preload capacity; and

- ability to under-ream (or “bell”) the anchor hole, to permit the expanded base of the anchor to mechanically engage the surrounding concrete under a preload condition.

Notwithstanding the above, it is recommended that expansion anchors should not be used in areas where cracking of the surrounding concrete is likely to occur.

In cases where expanding anchors are loaded dynamically or in static tension, it is recommended that the loading not exceed a proportion of the first-slip load, typically 65%. The first-slip load is commonly defined as the load at 0.1 mm slip. These particular uses for expanding anchors should be discussed, prior to use, with the manufacturer/supplier of the proprietary anchor.

7.6.6 Cast-in Steel Sections

The resistance of cast-in sections to shear and bending can be calculated assuming the formation of concrete compression stress blocks to resist the actions, and with due regard to the rigidity of the section as shown in Figure 7.4.

Tension forces should be resisted by anchor lugs that will mobilise the strength of a cone of concrete. Design will then be in terms of the principles previously outlined.

A recommended design approach is included in Example 2 of Appendix E1.

7.6.7 Influence of Cracking

The performance of embedments is adversely affected by cracking [7.4]. It is recommended that if an anchor is located in an area where cracking is likely to occur (from any cause) extra precautions should be taken, such as additional reinforcement to transfer the loads on the embedment back to an uncracked region of concrete.

It is not advisable to locate embedments in potential plastic hinge zones where extensive cracking can be expected and from where cover concrete may be lost. If relocation is not possible, the embedment should be designed neglecting the tensile and compression strength of the concrete. The load transfer should rely on truss action using transverse reinforcement to transfer tension where required, noting that the core concrete can also become badly cracked.

The recommended approach is to embed the anchors as deeply as possible into the core, and to ensure the potential failure surface engages the transverse rein-

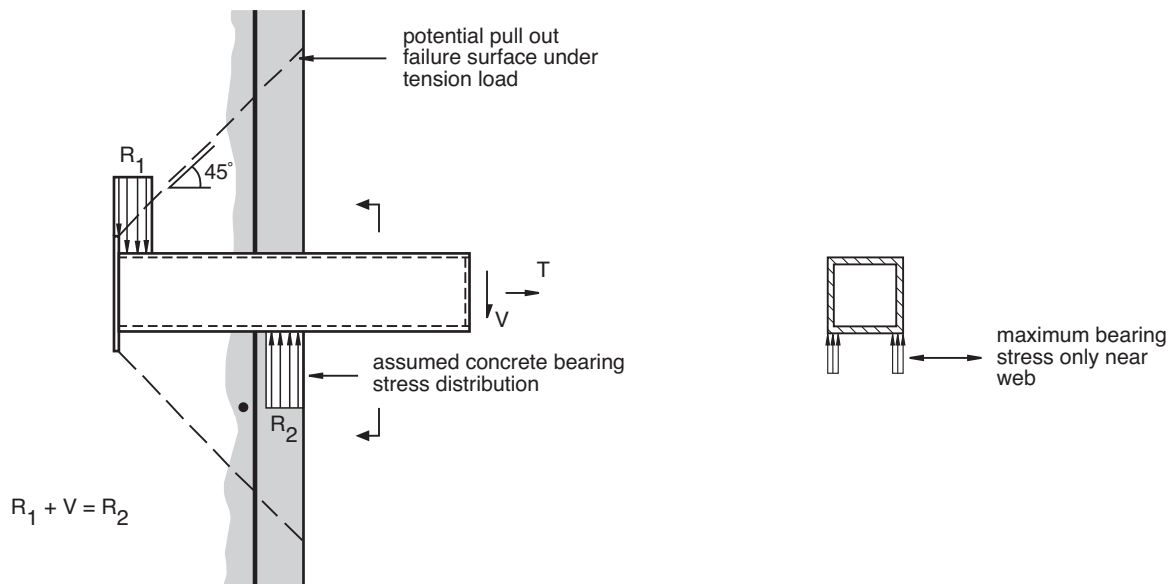


Figure 7.4: Actions on an embedded length of RHS

forcement in that region. It is recommended that post-drilled anchors should not be used in potential plastic hinge regions.

Fixings anchored in zones of flexural cracks should have their failure cones modified to account for the strength reduction of the embedment caused by the cracks. Reference 7.4 recommends that the resistance of the concrete mechanism be reduced in proportion to the crack widths. Refer to Section 7.6.8 for the influence of cracking on the design of welded plates with headed studs.

7.6.8 Detailing for the Effects of Spalled Concrete Cover

The design of embedments will normally include the contribution of all concrete surrounding the anchorage, including that in the cover zones.

Where embedments are unavoidably located in zones where spalling of cover concrete is likely, additional precautions are required as follows:

- The type of embedment selected must still permit the applied loads to be resisted by the assembly (albeit at reduced load factors), without the cover concrete forming part of the load transfer mechanism. This could be by flexure and tension in studs anchored into confined concrete, rather than by bearing on concrete.
- Anchorage beyond the cover zone should include allowance for concrete cracking. This requirement may preclude the use of headed studs, for example, in favour of longer reinforcing bars with hooked anchorage into the confined concrete.

- The type of embedment specified must exhibit load-deformation characteristics that are inherently ductile, i.e. not highly sensitive to assembly deformation, or changes in position of applied loads which may occur concurrently with the spalling of cover concrete.
- Special attention is required in the welding of galvanised and structural steel, to ensure that premature (and brittle) failure arising from weld zone embrittlement is avoided.

7.6.9 Testing of Embedded Steel Connections

The strength of embedded steel connectors may be determined by means of load testing no less than 10 (ten) specimens to failure. The test loads should simulate the most unfavourable combination of loads and forces likely to be applied to the embedded steel connector when in service. The design is the load at which no specimens fail. Less than 10 (ten) specimens can be tested if full account is taken of the effects of statistical variation.

7.7 Detailing of Steel Embedments in Concrete

7.7.1 General

In the case of overload, the embedments must yield or slip before failure, thus having sufficient ductility or flexibility to preclude a brittle concrete pull-out failure, or brittle fracture at or near the welds. The embed-

ment must also yield or slip in the attached connecting hardware before the component of the fixing anchored into the concrete can fail in a non-ductile manner. Where failure of the concrete resisting mechanism might occur, embedments should be attached to, or hooked around, reinforcing steel to effectively transfer forces to the reinforcing and surrounding concrete (see Figure 7.5).

7.7.2 Detailing for Limited Edge Distance

Details for this situation are provided in Reference 7.6. The design approach is to transfer the force from the embedment to reinforcing steel, which is anchored into the concrete core.

Tests on the shear resistance of anchor bolts under reversed cyclic loading [7.6], indicate that for small edge distances the fixing's performance can be adequate with the provision of two reinforcing steel hairpins or U-bars (Figure 7.5).

While the Type 1 detail (Figure 7.5(a)) provided better protection against concrete spalling at lower loads, it was observed that cyclic loading could cause concrete between the hairpin and the bolt to spall away. It is therefore recommended that for seismic loads the Type 2 detail be used.

7.7.3 Welding to Embedded Plates

When welding anchors or reinforcing bars to plates (Figures 7.1 (a) and (b)), plate laminar tearing must be prevented. It is recommended that a "pass-through and plug weld" detail be used (Figure 7.6).

To avoid concrete spalling at the edges of the plate when heat from on-site welding causes expansion, a gap between plate and concrete should be formed. The plate thickness should also be sufficiently thick to minimise welding distortions.

7.7.4 Corrosion Protection

Steel embedments are often vulnerable to corrosion, and are normally specified either hot-dipped galvanised, or with stainless steel components.

The following points should be noted when specifying corrosion protection:

- hot-dipped galvanising of mild steel can engender embrittlement in cold-worked sections, reducing ductility; and
- stainless steel, where attached to mild steel by welding, can promote crevice corrosion, or galvanic corrosion, in the contact areas.

Both of these phenomena are discussed in Appendix D2.

7.8 Recommendations

7.8.1 Steel Embedments

Steel embedments in all situations should be designed and detailed to suppress concrete pull-out failure, thereby ensuring a ductile mechanism.

7.8.2 Design and Detailing of Embedments

Steel embedments selected for a specific application should be evaluated on their ability to:

- withstand all imposed deformations, and applied loads; and
- accommodate structural damage, e.g. by spalling of cover concrete without loss of strength below an acceptable level.

7.8.3 Design Methods

Designers should check that design method assumptions, e.g. in source documents, are valid for the installed condition of the steel embedment. Specific references are recommended for use in particular applications.

7.8.4 Corrosion

Designers should specify embedments with a corrosion resistance appropriate to the design life of the structure and the envisaged frequency of maintenance inspections.

7.8.5 Use of Overseas Design Information in New Zealand

New Zealand designers should review and modify the design information contained in the listed references, to ensure that the approach used to determine structural strength is consistent with that adopted in other New Zealand standards.

7.9 References

- 7.1 *Code of Practice for General Structural Design and Design Loadings for Buildings*, NZS 4203:1992, Standards New Zealand, Wellington, 1992, 45 pp.
- 7.2 *Concrete Structures Standard*, NZS 3101:1995, Standards New Zealand, Wellington, 1995, 256 pp.

- 7.3 “Mechanical fasteners for concrete”, *Information Bulletin IB31*, Cement and Concrete Association of New Zealand, Porirua, 10 pp.
- 7.4 *Fastenings to Concrete and Masonry Structures - State of the Art Report*, Comité Euro-International du Béton, Thomas Telford, 1994, 249 pp.
- 7.5 Restrepo-Posada J I and Park, R. “Tensile capacity of steel connectors with short embedment lengths in concrete”, *Research Report 93-6*, Department of Civil Engineering, University of Canterbury, August 1993, 51 pp.
- 7.6 Klingner R E, Mendonca, J A and Malik, J B. “Effect of reinforcing details on the shear resistance of anchor bolts under reversed cyclic loading”, *ACI Journal*, Vol 79 No 1, Jan-Feb 1982, pp. 3-12.
- 7.7 Hsu T T C. “Fatigue of plain concrete”, *ACI Journal*, Vol 78 No.4, July-August 1981, pp. 292-305.
- 7.8 Brown, B J, Vautier E W and Shepherd, D A. “Grouting of anchors and reinforcing starter bars into concrete”, *Research Project No. 108905 (Draft Report)*, Transit NZ, Wellington, May 1990, 69 pp.

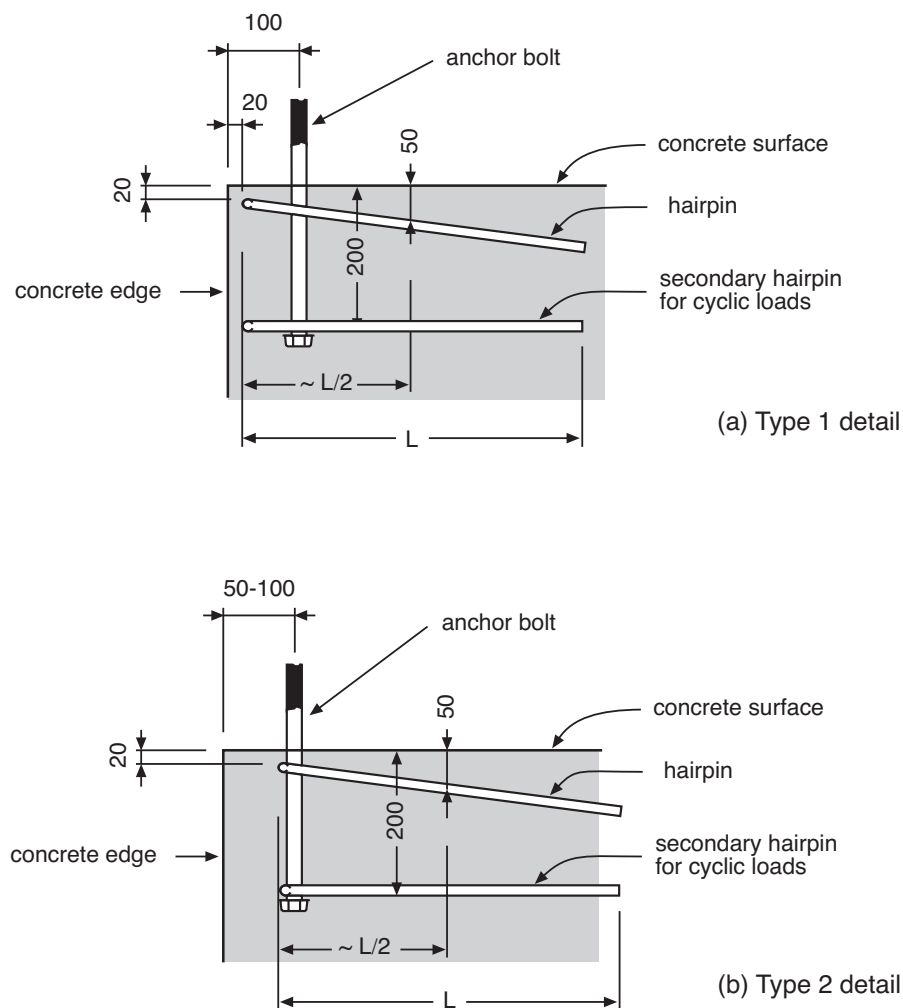


Figure 7.5: Hairpin reinforcement to embedded bolt with small edge distance

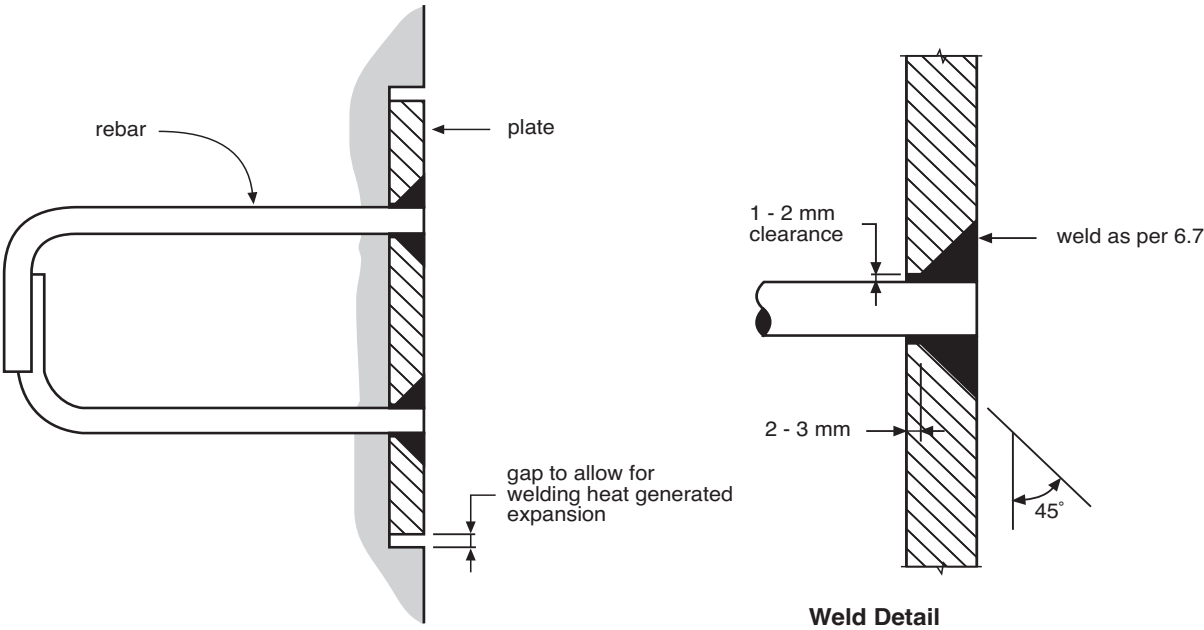


Figure 7.6: Connection of reinforcing bars to plate

Chapter 8

Tolerances

8.1 Introduction

Successful precast concrete construction relies on a full understanding of the need for adequate tolerances and the full implications of variations in dimensions. This understanding must be developed by designers, fabricators and constructors.

The requirements regarding tolerances for precast concrete construction are contained in the New Zealand Standard NZS3109 1997 [8.1].

The 1997 version of this Standard was derived from studying Australian and American sources, coupled with the discussions raised in Chapter 8 of the 1992 edition of this publication. The revised tolerances are very similar to the provisions contained in the Prestressed Concrete Institute (PCI) Committee on Tolerances [8.2] and the subsequent incorporation of this material into a more general publication [8.3].

In the preparation of a national code such as NZS3109, there remains always the difficulty of selecting appropriate information for all eventualities. This can often lead to significantly over-complicated documentation. Given that PCI examples of tolerances provided significant illustration of the combination of tolerances from manufacture and erection, the PCI examples of Figures 8.1 (a) and (b), 8.2, 8.3 and 8.4 have been retained. To help the designer, however, tolerances from the New Zealand Standard 3109 Tables 5.1, “Tolerances for precast components” and 5.2, “Tolerances for in situ construction” have been substituted for the PCI values where similar definitions occur.

It is essential that the designer and precaster understand the tolerance requirements of NZS3109 because, by reference, all work designed to NZS3101 is deemed to be constructed in accordance with NZS3109.

The approach in this section is to begin by considering tolerances as three different types (namely product, erection, and interface tolerances) and then to look at the question of clearances.

8.2 Product Tolerance

Product tolerances relate to the dimensions of an individual component. They are set by the designer to

control production in order to achieve the structural and architectural requirements. They should be as large as possible, but commensurate with the nature and type of structure being built. Close tolerances increase costs, especially where limited production numbers are required.

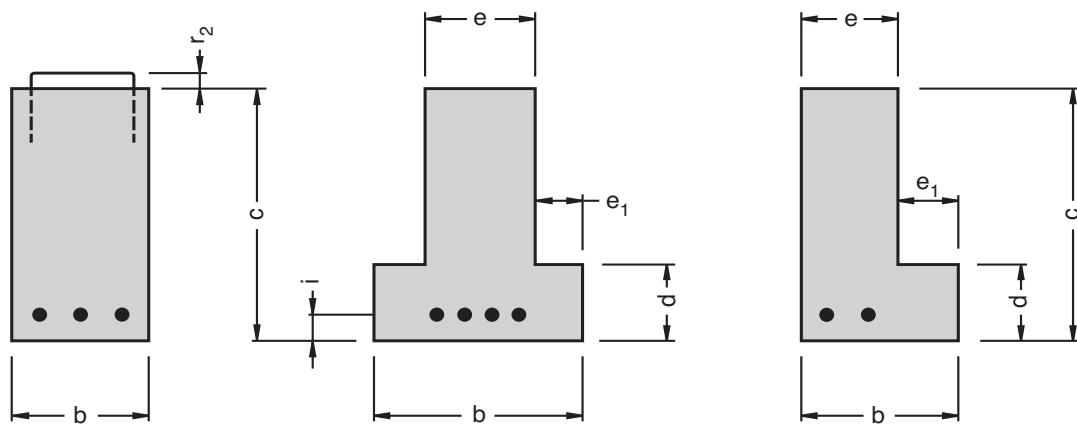
Product tolerances for New Zealand are given in Table 5.1 of NZS3109. An example of PCI, American product tolerances [8.2], is given in Figures 8.1 (a) and (b). The comparative New Zealand tolerance figures, where available, have been added to the table. These tolerances are generally accepted by precast concrete manufacturers and may be used. However, designers should always check the appropriateness of a given tolerance for each particular situation. In some cases the recommended tolerance may not be suitable.

8.3 Erection Tolerance

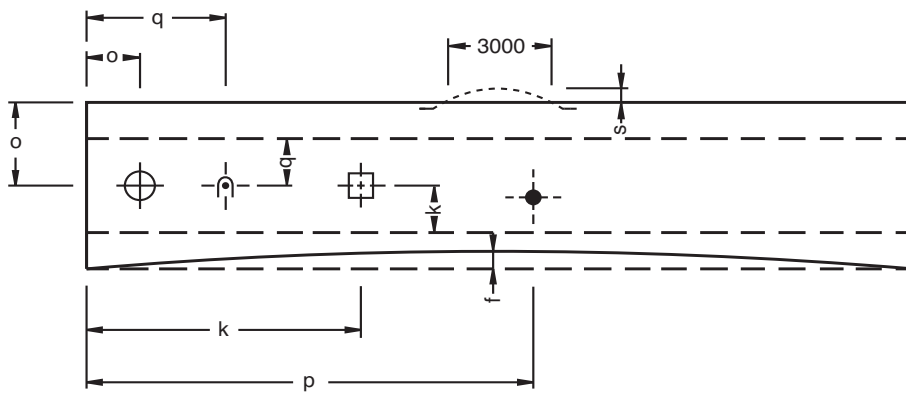
Erection tolerance is the allowance between actual component location and primary control surfaces such as grids, datum levels, etc. Some examples of PCI erection tolerances [8.2], which are generally achievable by the New Zealand construction industry, are given in Figures 8.2 to 8.5. If smaller tolerances are required, increased building costs can be expected.

However, reference *must* be made to NZS3109 Table 5.2, “Profile tolerances”, which deal with position within a structure and plumbness of the structure. Tolerance differences between the two references are small, sometimes lower, sometimes higher, e.g. Plan position PCI Structural +13 mm, Architectural ~10 mm NZS3109 ~10 mm. Where precast concrete is to be fitted to a structural steel frame, then the erection tolerances from NZS3404 Clause 15.3 need to be taken into account.

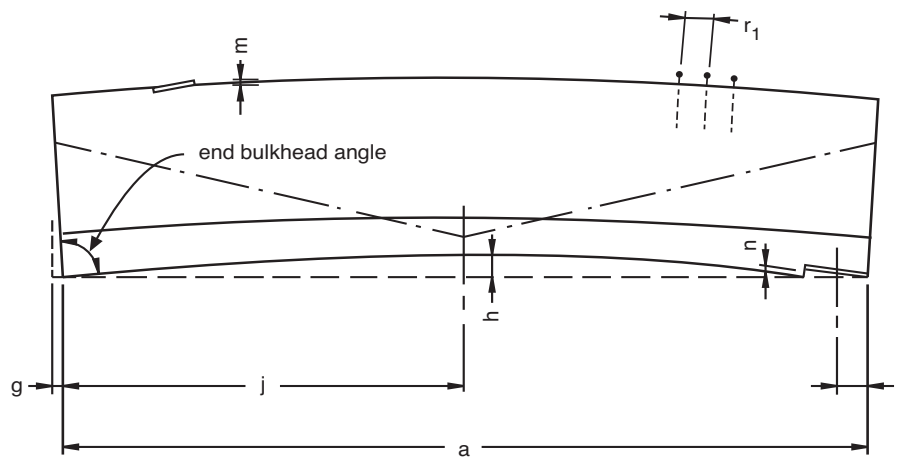
The use of “primary control surfaces” [8.2] are considered a useful technique for specifying erection tolerances. It is the positional dimensions of these surfaces that are controlled during erection. The positional dimensions of all other parts of the component will be governed by the erection and product tolerances. For example, consider the beam-column junction in Figure 8.3. The beam bearing surface for the precast



Cross Sections



Plan



Elevation

Figure 8.1(a): Product tolerances for building beams and spandrels.
(From Reference 8.2)

(This, and other figures from Reference 8.2 are included with permission of PCI).

concrete floor units is a control surface, not the top of the column.

The accuracy achieved in constructing the foundations of buildings has a major bearing on the success or otherwise of the assembly of precast concrete units. The contractor must therefore establish appropriate

surveying controls. Before erection commences, after erection, and before other trades start work, the accuracy of erection should be verified. Failure to use these controls can, and frequently does, lead to costly and time consuming alterations and arguments.

Like product tolerances, erection tolerances should be

			†NZS3109 from Table 5.1 mm	
a	=	Length	±19 mm	Varies with length ±8 to ±20
b	=	Width (overall)	±6 mm	Varies with length ±5 to ±10
c	=	Depth (overall)	±6 mm	Varies with length ±5 to ±10
d	=	Depth (ledge)	±6 mm	±5
e	=	Stem width	±6 mm	Varies ±5 to ±10
e ₁	=	Ledge width	±6 mm	±5
f	=	Sweep (variation from straight line parallel to centerline of member)		Varies with length 3 to
		Up to 12m member length	±6 mm	L/1000 or 15 max
		12 to 18m member length	±13 mm	
		Greater than 18m member length	±16 mm	
g	=	Variation from specified end squareness or skew	±3 mm per 300 mm depth, ±13 mm max	Varies with length ±5 to ±15
h*	=	Camber variation from design camber	±3 mm per 3 m, ±19 mm max	Varies with length ±5 to ±15
i	=	Position of tendons		±5
		Individual	±6 mm	
		Bundled	±13 mm	
j	=	Longitudinal position from design location of deflection points for deflected strand		
		Member length 9m or less	±150 mm	
		Member length greater than 9m	±300 mm	
k	=	Position of plates	±25 mm	±20
l	=	Position of bearing plates	±13 mm	Inserts ±12
m	=	Tipping and flushness of plates	±6 mm	±5
n	=	Tipping and flushness of bearing plates	±3 mm	±5
o	=	Position of sleeves both horizontal and vertical plane	±25 mm	±12
p	=	Positions of inserts for structural connections	±13 mm	±12
q	=	Position of handling devices		
		Parallel to length	±300 mm	±12
		Transverse to length	±13 mm	±12
r	=	Position of stirrups		See Clause 3.9 NZS3109
		r1 longitudinal spacing	±50 mm	
		r2 projection above surface of beam	±6 mm - 13 mm	
s	=	Local smoothness any surface	(6 mm in 3 m)	See NZS3114
		Does not apply to top surface left rough to receive a topping or to visually concealed surfaces		

* For members with a span to depth ratio approaching or exceeding 30, the stated camber tolerance may not apply. If the application requires control of camber to this tolerance in beams of such span-to-depth ratios, special premium production measures may be required. This requirement should be discussed in detail with the producer.

† Table 5.1 also has further information for precast slabs and additional fitments in the precast units.

Figure 8.1 (b): Product tolerances for building beams and spandrels
(From Reference 8.2)

as liberal as possible for the efficient matching of the units. It is recommended that, wherever possible, the designer review the proposed tolerances with the manufacturers and erectors prior to deciding on the final project tolerances.

8.4 Interface Tolerance

Interface tolerance refers to the allowance needed for the jointing or attaching materials in contact with the precast units. However, the immediate area of interest is the interfacing between precast units themselves and between precast units and cast-in-place concrete. This interfacing has often caused problems in the past.

One common example is the clashing of reinforcing bars at a beam-column junction. Often, the position of the projecting column steel is such as to displace the beam from its true position. A possible consequence of this is the loss of bearing of the precast floor units supported on one side of the beam, and too much bearing on the other side. Displacement of the beam by this type of interfacing problem also has a cumulative effect, especially where long beams span two or three bays of the frame. Frequently, the lapping of reinforcing bars at midspan beam joints is only achieved with the aid of gas sets or hydraulic jacks.

8.5 Clearances

The provision of clearances is recognition of the need for interface tolerances. Clearance should provide a buffer area where combined erection and production variations can be absorbed, and the actual clearance provided should reflect all the previously specified tolerances.

If allowance is made for the worst absolute algebraic combination of permissible tolerances, namely product tolerance + erection tolerance (including the tolerance of cast-in-place concrete) + interface tolerance, then it can become virtually impossible to detail interface connections. For example, consider the seating of precast concrete floor units on a precast beam, as shown in Figure 8.6(a). If all the maximum tolerances from NZS 3109 are used, the cumulative tolerance for the seating is + 36 mm. This would require a nominal design seating of approximately 101 mm, which is far more than currently used in practice. This example indicates a need to review the current requirements of NZS 3109. The reality is that many contractors have successfully built structures to far more stringent tolerances.

The recommended method to cope with this problem is for the designer to indicate in the contract documents

(preferably the working drawings) where tighter tolerances than those specified in NZS 3109 are required. The provided clearances should also be indicated so the contractor can make appropriate allowances during both tendering and construction. In many instances the contractor may be required to use special templates to ensure accurate starter locations for reinforcing bars, etc.

Two approaches have been suggested to combine permissible tolerances to calculate clearances. Reference 8.2 emphasises the need for engineering judgement to assess the likelihood of maximum product tolerances occurring in one location. The second approach [8.4] uses a statistical method in which the combined tolerance is the square root of the sum of the individual tolerances squared. This method is not only suitable where there is insufficient previous experience as a basis for sound judgement, but it will also give designers a realistic value for the required clearance. An example is given in Figure 8.6(b).

Designers are referred to Reference 8.2 which provides examples of calculation of clearances for several different construction situations.

8.6 Implications for Design

When designing connection details involving precast members, it is important that designers allow for increased structural actions due to the occurrence of unfavourable construction tolerances. The effect of tolerances will usually be negligible on the design of structural elements such as precast concrete floor units. However, when the design of a connection is largely influenced by the value of a small dimension which is sensitive to variation caused by tolerances, then tolerances should be allowed for in the design. An example is shown in Figure 8.7, where a steel angle is designed to support a precast concrete floor unit. The design clearance between the end of the bolt and the unit is 20 mm, but when cumulative permissible deviations are considered, the actual gap is increased by 20 mm, resulting in significantly increased stresses in the bolt and steel angle.

It is vital when designing connections where tolerances are significant, that cumulative tolerances be used to determine the worst design case.

8.7 Recommendations

- Tolerances from NZS3109 from Table 5.1 and 5.2 should be used as the primary reference points, but references 8.2 and 8.3 should be used as recommended resource documents where special additional concerns may need to be addressed.

Where precast concrete is to be attached to a structural steel frame, reference is also needed to NZS3404, Clause 15.3.

- Designers should liaise as closely as possible with contractors when specifying tolerances so appropriate allowances can be made, thereby reducing construction difficulties (Section 8.5).
- It is recommended that when designing connections that are sensitive to tolerances, cumulative

maximum permissible tolerances be used to define the worst design case (Section 8.6).

8.8 References

8.1 *Specification for Concrete Construction, NZS 3109:1997*, Standards Association of New Zealand, Wellington, 1997, 67 pp.

8.2 PCI Committee on Tolerances, "Tolerances for

a	=	Plan location from building grid datum	
		Structural applications	±13 mm
		Architectural applications	±10 mm
b	=	Top elevation from nominal top elevation	
		Maximum low	±13 mm
		Maximum high	±6 mm
c	=	Bearing haunch elevation from nominal elevation	
		Maximum low	±13 mm
		Maximum high	±6 mm
d	=	Maximum plumb variation over height of element (element in structure of maximum height 30 m)	± 25 mm
e	=	Plumb in any (3 m) of element height	±6 mm
f	=	Maximum jog in alignment of matching edges	
		Architectural exposed edges	±6 mm
		Visually non-critical edges	±13 mm

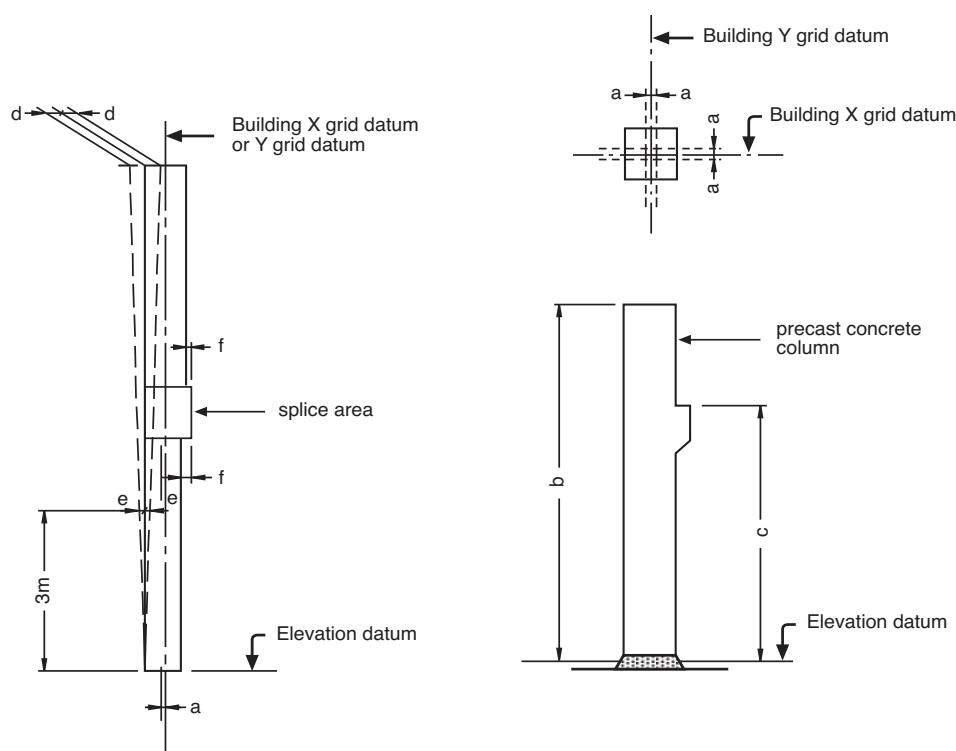


Figure 8.2: Erection tolerances for columns (Reference 8.2)

precast and prestressed concrete”, *PCI Journal*, January-February 1985, pp. 26-112.

Concrete, Prestressed Concrete Institute, Chicago, 1985, 87 pp.

8.4 *Code of Practice for Accuracy in Building BS 5606:1978*, British Standards Institution, London, 59 pp.

8.3 Recommended Practice for Erection of Precast

a	=	Plan location from building grid datum	±25 mm
a ₁	=	Plan location from centreline of steel*	±25 mm
b	=	Bearing elevation** from nominal elevation at support	
		Maximum low	13 mm
		Maximum high	6 mm
c	=	Maximum plumb variation over height of element	
		Per 300 mm height	3 mm
		Maximum	13 mm
d	=	Maximum jog in alignment of matching edges	
		Architectural exposed edges	6 mm
		Visually non-critical edges	13 mm
e	=	Joint width	
		Architectural exposed edges	±6 mm
		Hidden joints	±19 mm
		Exposed structural joint <i>not</i> visually critical	±13 mm
f	=	Bearing length*** (span direction)	±19 mm
g	=	Bearing width***	±13 mm

* For precast elements on a steel frame, this tolerance takes precedence over tolerance on dimension “a”.

** Or member top elevation where member is part of a frame without bearings.

*** This is a setting tolerance and should not be confused with structural performance requirements set by the architect/engineer. the nominal bearing dimensions and the allowable variations in the bearing length and width should be specified by the engineer and shown on the erection drawings.

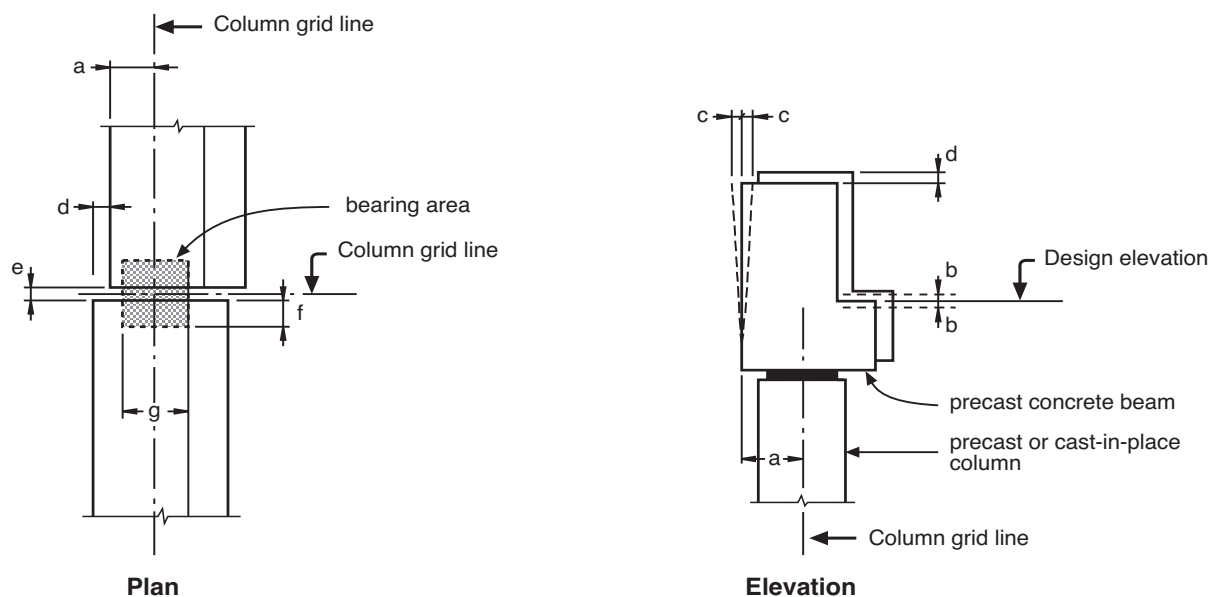


Figure 8.3: Erection tolerances for beams and spandrels, precast element to precast concrete, cast-in-place concrete, masonry or structural steel (From Reference 8.2)

a	=	Plan location from building grid datum	±25 mm
a ₁	=	Plan location from centreline of steel*	±25 mm
b	=	Top elevation from nominal top elevation at member ends	
		Covered with topping	±19 mm
		Untopped floor	±6 mm
		Untopped roof	±19 mm
c	=	Maximum jog in alignment of matching edges (both topped and untopped construction)	±25 mm
d	=	Joint width	
		Member length 0 - 12 m	±13 mm
		12.1 - 1.8 m	±19 mm
		18.1 m plus	±25 mm
e	=	Differential top elevation as erected	
		Covered with topping	±19 mm
		Untopped floor	±6 mm
		Untopped roof**	±19 mm
f	=	Bearing length*** (span direction)	±19 mm
g	=	Bearing width***	±19 mm
h	=	Differential bottom elevation of exposed hollow-core slabs****	±6 mm

* For precast concrete erected on a steel frame building, this tolerance takes precedence over tolerance on dimension "a".

** It may be necessary to feather the edges ±6mm to properly apply some roof membranes.

*** This is a setting tolerance and should not be confused with structural performance requirements set by the architect/engineer. The nominal bearing dimensions and the allowable variations in the bearing length and width should be specified by the engineer and shown on the erection drawings.

**** Untopped installations will require a larger tolerance.

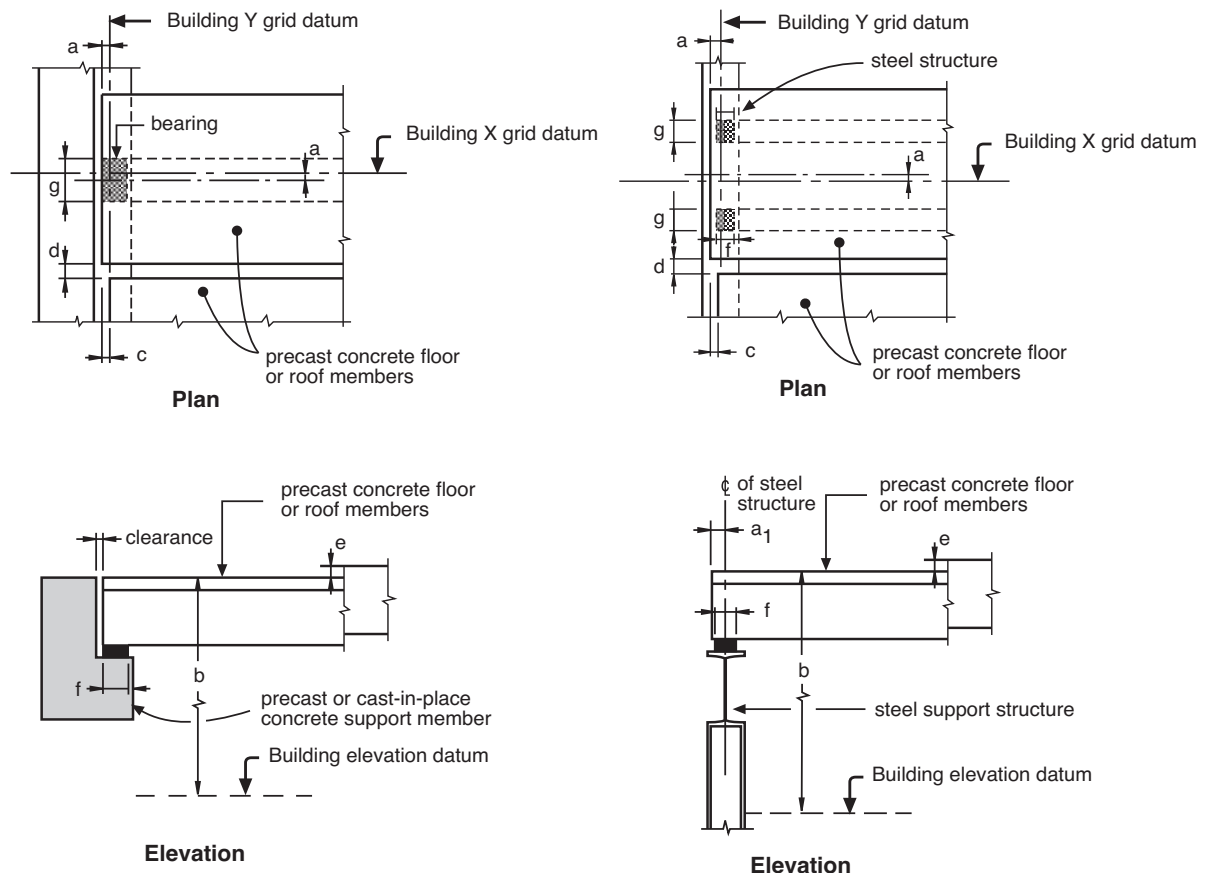


Figure 8.4: Erection tolerances for floor and roof members (Reference 8.2)

a	=	Plan location from building grid datum*	±25 mm
a ₁	=	Plan location from centreline of steel**	±25 mm
b	=	Top elevation from nominal top elevation	
		Exposed individual panel	±13 mm
		Nonexposed individual panel	±19 mm
		Exposed relative to adjacent panel	±13 mm
		Nonexposed relative to adjacent panel	±19 mm
c	=	Bearing elevation from nominal elevation	
		Maximum low	13 mm
		Minimum high	6 mm
d	=	Maximum plumb variation over height of structure or 30 m whichever is less**	25 mm
e	=	Plumb in any 3 m of element height	6 mm
f	=	Maximum jog in alignment of matching edges	13 mm
g	=	Joint width (governs over joint taper)	±10 mm
h	=	Joint taper over length of panel	13 mm
h ₃	=	Joint taper over 3 m length	10 mm
i	=	Maximum jog in alignment of matching faces	
		Exposed	10 mm
		Nonexposed	19 mm
j	=	Differential bowing, as erected, between adjacent members of the same design	13 mm

* For precast buildings in excess of 30m height, tolerances “a” and “d” can increase at a rate of 3 mm per storey to a maximum of 50 mm.

** For precast concrete erected on a steel frame building, this tolerance takes precedence over tolerance on dimension “a”.

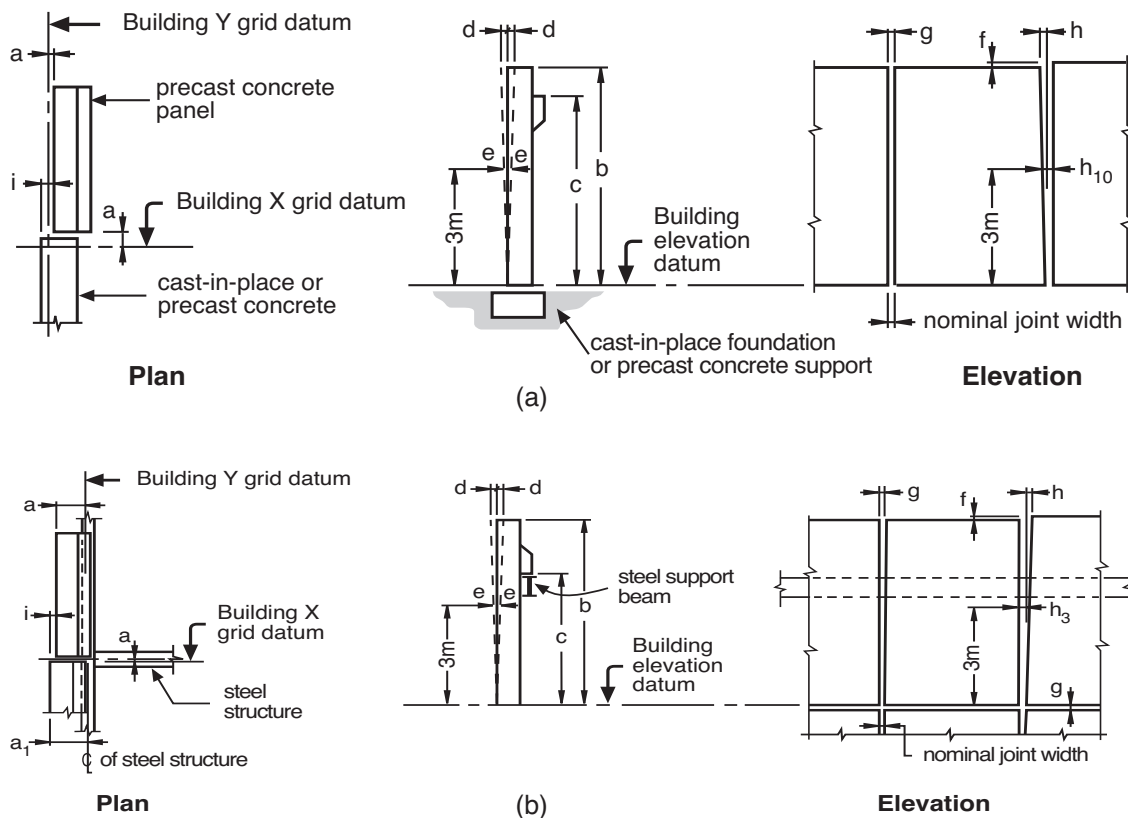
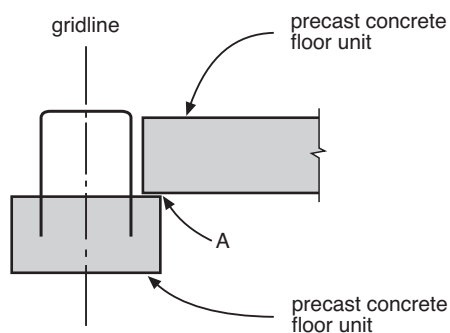


Figure 8.5: Erection tolerances for structural wall panels (a) precast element to precast or cast-in-place concrete or masonry, (b) precast element to structural steel (Reference 8.2)



Tolerances for point A

± 10 position in plan Table 5.2
 ± 2.5 cross-sectional dimension ($50\% \pm 5$)
 ± 8 straightness ($L = 8$ m)
 total ± 20.5

Tolerances for floor unit

± 10 length ($L > 12$) ($50\% \pm 20$)
 ± 5 squareness
 total ± 15

Maximum setting width required
 = 65 (design seating) + 20.5 + 15
 = **100.5 mm**

(a) Combination by absolute
algebraic addition

Tolerances for point A

$$(10^2 + 2.5^2 + 8^2)^{0.5} = 13.0$$

Tolerances for floor unit

$$(10^2 + 5^2)^{0.5} = 11.2$$

Maximum setting width required
 = 65 + $(13^2 + 11.2^2)^{0.5}$
 = **82.2 mm**

(b) Combination by square root of
the sum of the squares

Figure 8.6: NZS3109 tolerances applied to a precast beam-floor unit interface using two combination members

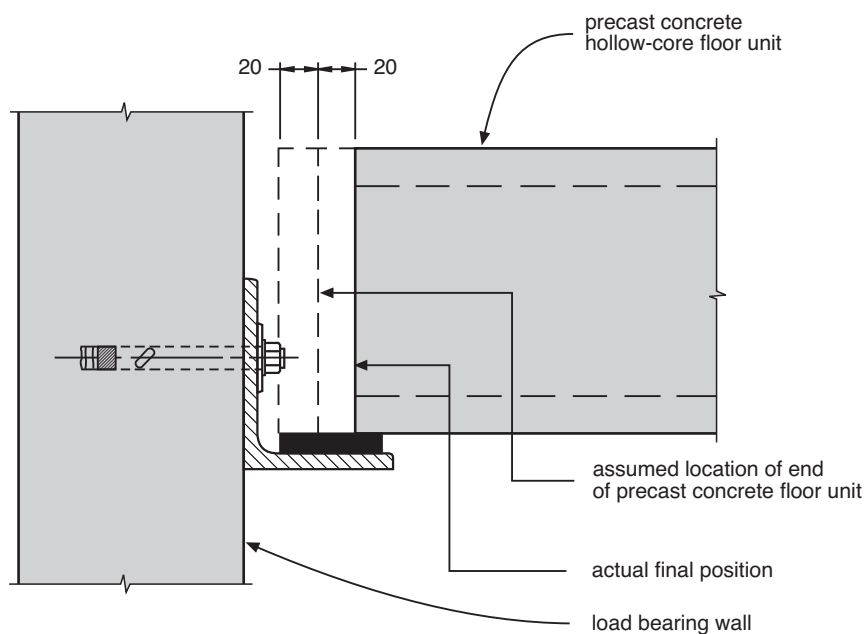


Figure 8.7: Precast concrete floor unit - structural steel seating detail

Appendix A1: Beam Elongation due to Plastic Hinging

Elongation occurs with the formation of a plastic hinge in a member. This arises as the rotation occurs predominantly due to tensile yielding of the reinforcement rather than the crushing of the concrete. Longitudinal extensions in beams of the order of 2 to 4 percent of the beam depth per plastic hinge have been observed in tests in which expansion was free to occur [A1.1 to A1.5]. In a test of a combined frame-wall structure, the elongation of the wall was found to result in a redistribution of internal action and a major increase in lateral strength of the system [A1.6].

In a uni-directional plastic hinge in a beam, that is a hinge in which inelastic deformation occurs in only one direction in an earthquake (see Figure A1.1 (a)), or in the first inelastic displacement in a reversing plastic hinge, the compressive strains in the compression reinforcement are small and for practical purposes may be neglected. On this basis the growth in length in each hinge zone may be calculated from the expression:

$$\text{extension} = \frac{\theta(d-d')}{2} \quad (\text{A1.1})$$

where θ is the rotation of the plastic hinge zone and $d - d'$ is the distance between the centroids of top and bottom reinforcement in the beam [A1.3].

With reversing plastic hinge zones it has been found that cracks in the compression zone do not close unless axial forces appreciably greater than the maximum shear force are applied to the member [A1.4 and A1.5]. In the absence of a restraining force, the member continues to elongate as cyclic inelastic deformations are applied. Test results have indicated that the elongation in a typical reversing plastic hinge in a beam, which have been subjected to two complete cycles of ± 2 and ± 4 displacement ductilities and then taken to the first displacement ductility of 6, is approximately twice the value given by equation A1.1. For this situation, as illustrated in Fig A1.1, the plastic hinge rotation can be assessed from the expression:

$$\theta = \frac{\Delta}{hk} \quad (\text{A1.2})$$

where Δ is the interstorey deflection, h is the interstorey height and k is the ratio of the distance between the hinge zones in a beam to the column centrelines.

On this basis the elongation in the length of a structure, which is not restrained against expansion and which

forms reversing hinges in a major earthquake, is approximately given by:

$$\text{extension} = \frac{2}{k} n \frac{\Delta}{h} (d - d') \quad (\text{A1.3})$$

where n is the number of bays.

For the case where uni-directional plastic hinges form in a severe earthquake, as illustrated in Fig 1.1(b), the inelastic rotations accumulate with the passage of the earthquake. The resultant magnitude of plastic hinge rotation depends upon the duration of strong ground shaking. Several series of analyses have been made with earthquake records with intensities and durations similar to that envisaged in the loadings code [A1.2, A1.8]. From these, it was found that the plastic hinge rotation was typically 2 to 4 times the value for a reversing plastic hinge as given by Eq.A1.2.

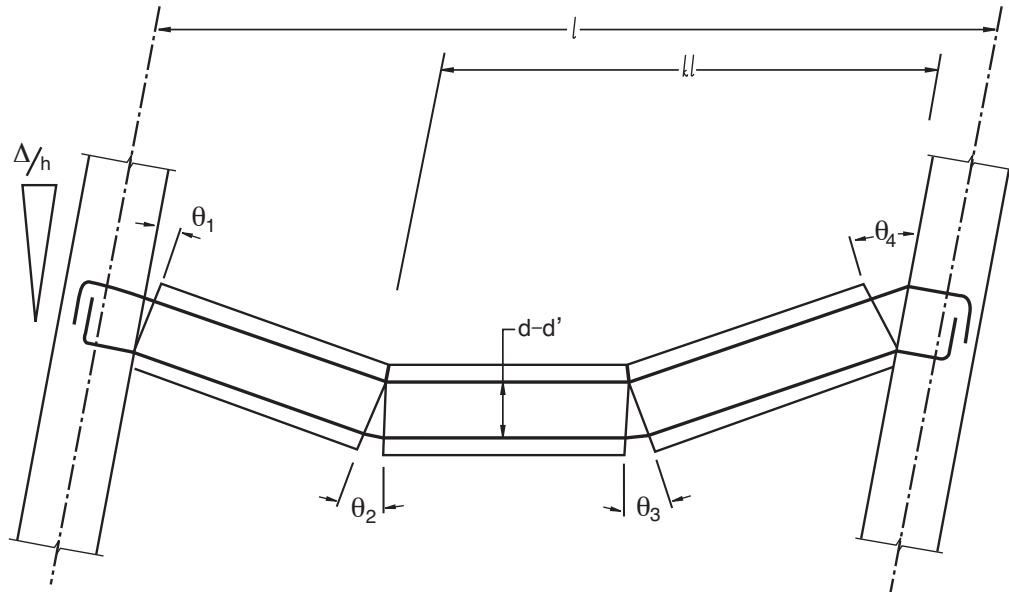
The elongation caused by tensile yielding of reinforcement associated with plastic hinge formation can be readily visualised by drawing the plastic hinge zones as concentrated areas as illustrated in Figures A1.1 and A1.2. As indicated in Figure A1.2, particularly severe effects can arise where the seismic resistance is provided by a perimeter frame. In this situation, the accumulated elongation from the beams in several bays may be imposed on a single bay of a precast floor system.

In calculating the maximum interstorey deflection during a severe earthquake, allowance should be made for the inelastic deformation that develops. For structures of limited ductility, when the structural ductility factor is 2 or less, the values of the maximum interstorey deflection can be estimated from the lateral deflections in an elastic analysis scaled by the structural ductility factor. However, in ductile high-rise moment-resisting frame structures, the critical interstorey deflection is typically 1.7 times as great as estimated by scaling elastic-based deflections by the structural ductility factor. This aspect is recognised by the code [A1.7], which gives different interstorey deflection limits depending upon whether an elastic-based analysis, or an inelastic-based time history analysis is used [A1.9, A1.10].

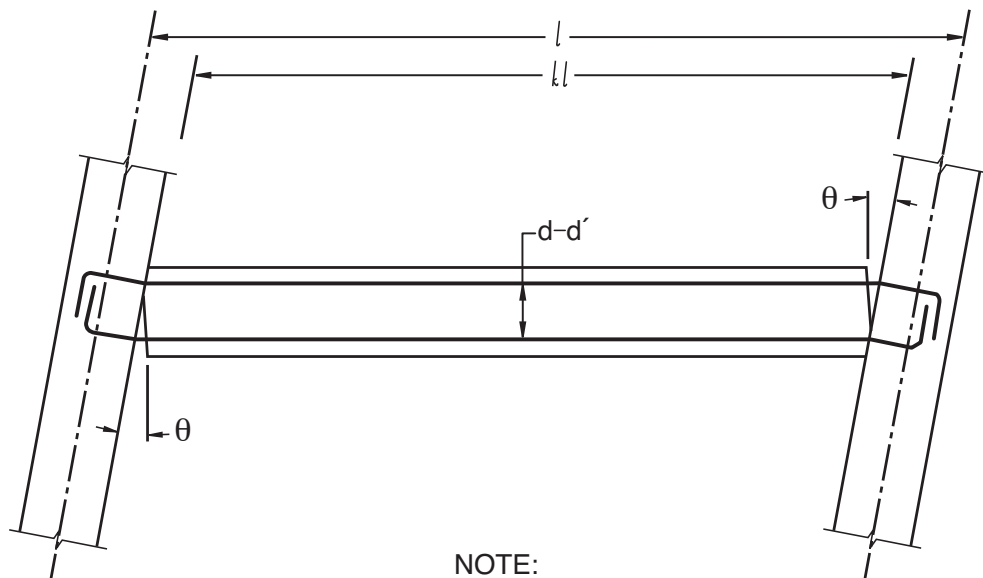
Further research is required to establish the magnitudes of member elongation that may occur in severe earthquakes. The recommendations in this appendix should be treated as tentative until this work has been carried out.

References

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- A1.5 Fenwick, R C and Davidson, B J. "Elongation in ductile seismic resistant RC frames", *Recent Developments in Lateral Force Transfer in Buildings*, American Concrete Institute Special Publication SP 157, 1995, pp 143-170.
- A1.6 Wight, J K (editor). *Earthquake Effects on Reinforced Concrete Structures*, American Concrete Institute, Special Publication SP84, 1985, 428pp.
- A1.7 *Code of Practice for General Design and Design Loadings for Buildings*, NZS 4203-1992, Standards New Zealand, Wellington, 1992.
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- A1.9 Paulay, T. "A consideration of p-delta effects in ductile reinforced concrete frames", *Bulletin of NZ National Society for Earthquake Engineering*, Vol 11, No.3., Sept 1978, pp 151-160.
- A1.10 Moss, P G and Carr, A J. "The effects of large displacement on the earthquake response to tall frames structures", *Bulletin of NZ National Society for Earthquake Engineering*, Vol 13, No.4, Dec 1980, pp 317-328.



(a) Uni-directional hinges in a bay



NOTE:

The hinge zones have been drawn as concentrated rotations at a section.

To allow for the spread of column face hinges the centre of rotation may be taken $\frac{(d-d')}{4}$ from the column face

(b) Reversing hinges in a bay

Figure A1.1: Reversing and uni-directional plastic hinges in a beam

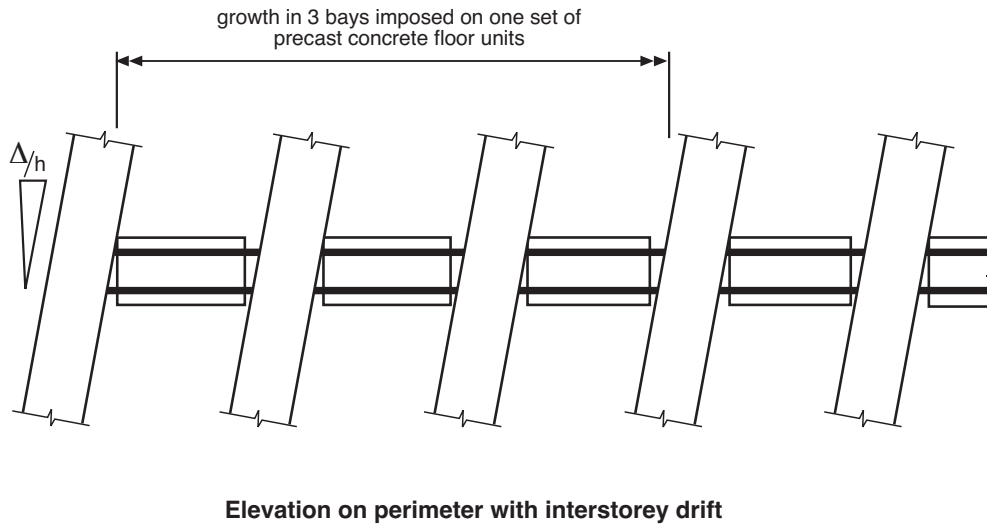
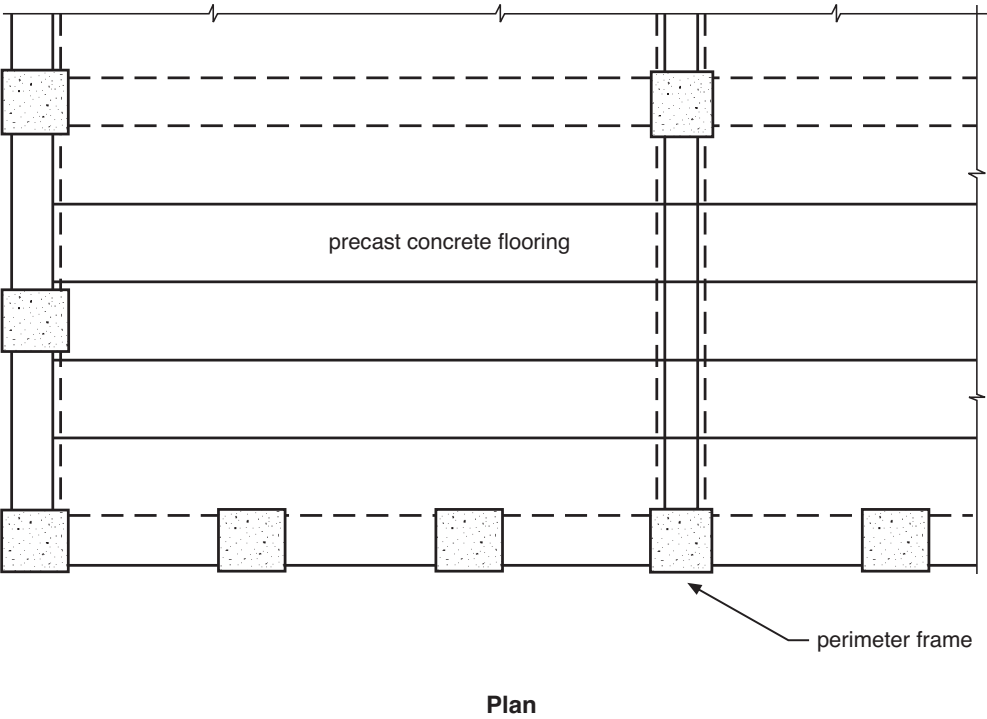


Figure A1.2: Growth in perimeter frame imposed on precast concrete flooring

Appendix A2: Allowances for Effects of Spalling

Material of Support	Distance Assumed Ineffective
Steel	0
* Concrete Grade 30 or over, plain or reinforced (in general)	15 mm
* Brickwork or masonry	25 mm
* Concrete below Grade 30, plain or reinforced (in general)	25 mm
Reinforced concrete less than 300 mm deep at outer edge	not less than nominal cover to reinforcement on outer face of support
Reinforced concrete where vertical-loop reinforcement exceeds 12 mm diameter	Nominal cover <i>plus</i> inner radius of bend
* Where unusual spalling characteristics are known to apply when particular constituent materials are being used, adjustment should be made to the distances recommended	

Table A2.1: Allowances for effects of spalling at supports
(From Reference 2.5, with permission)

Reinforcement at Bearing of Supported Member	Distance Assumed Ineffective
Straight bars, horizontal loops or vertical loops not exceeding 12 mm in diameter, close to end of member	10 mm or end cover, whichever is the greater
Tendons or straight bars exposed at end of member	0
Vertical loop reinforcement of bar size exceeding 12 mm in diameter	End cover <i>plus</i> inner radius of bar

Table A2.2: Allowances for effects of spalling at supported members
(From Reference 2.5)

Appendix B1: Precast Concrete Frame Connection for System 1 Design Bending Moments

With System 1 type connections, the precast beams may be supported at each column face so that the dead and construction live loads are initially carried by the beam acting as a simply supported member. When the cast-in-place concrete hardens, the initial simple supports for the beam become moment-resisting connections, which sustain bending moments in response to any rotation subsequently applied to them. The structural form has been changed. In its initial condition the initial dead load acting on the simple span causes rotations to occur at the supports. Due to creep and shrinkage in the concrete these rotations continue to increase with time. However, the hardening of the cast-in-place concrete and the resultant change in the structural form leads to a redistribution of that portion of the bending moment arising from the dead load applied to the member before the structural form was changed. Dead loads applied after the structure has been modified are not subject to redistribution.

With the redistribution of bending moments described above, the values change from those sustained in the initial state towards those that would be sustained if the load in question had been applied to the structure in its final condition. This is illustrated in Figure B1.1. How far this redistribution goes depends upon the creep and shrinkage characteristics of the concrete, the time between the initial condition and when the change in structural form is made, and the arrangement of reinforcement in the beam. Making a rational allowance for these factors is complex. Readers wishing to pursue this are referred to references B1.1 and B1.2. An approach by which the extent of redistribution can be assessed may be based on the method used in the concrete design standard [B1.3] for finding long term deflections. The long term deflection is given as K_{cp} times the short term deflection, where the factor K_{cp} is given by the equation:

$$K_{cp} = 2 - 1.2 \frac{A'_s}{A_s} > 0.6$$

This value corresponds to an effective creep coefficient for the beam as a whole, and it may be used with the effective modulus method to indicate the propor-

tions of the initial dead load resisted by the structure in its initial and final forms when creep has ceased. On this basis the proportion of the load carried by the initial form, p_{in} , is given by:

$$p_{in} = \frac{1}{1 + K_{cp}}$$

with the remainder of the load $\left[\frac{K_{cp}}{1 + K_{cp}} \right]$ being resisted by the structure in its final form.

In checking the performance of the beam for serviceability, it should be noted that this redistribution of bending moments occurs with time. Consequently, it may be necessary to check for both the initial and final distribution of bending moment.

In the strength limit state further redistribution may occur with the formation of plastic hinges in the beam.

The design and construction of beams propped only at the ends also raises the issue of the shear strength of the precast beam in its temporary construction state, as shown in Figure B1.2, since it could be considered that the stirrups are inadequately anchored. This issue has been already discussed in Section 3.2. However, no shear problems have been observed in practice during construction using this system.

References

- B1.1 ACI Committee 209, *Designing for Creep and Shrinkage in Concrete Structures*, American Concrete Institute, Special Publication, SP76, 1982.
- B1.2 Bryant, A H, Wood, J A and Fenwick, R C. "Creep and shrinkage in concrete bridges", *RRU Bulletin 70*, National Roads Board, Wellington, 1984.
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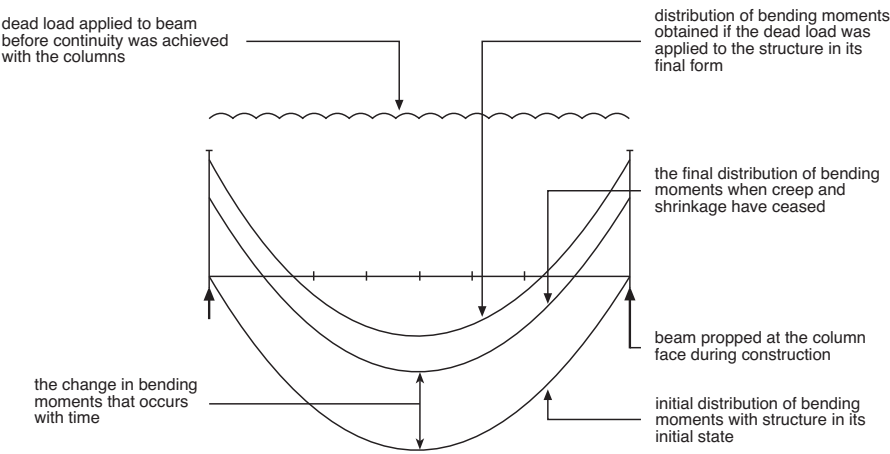


Figure B1.1: Redistribution of bending moments associated with creep and shrinkage where a load is applied to a structure which is subsequently changed in form

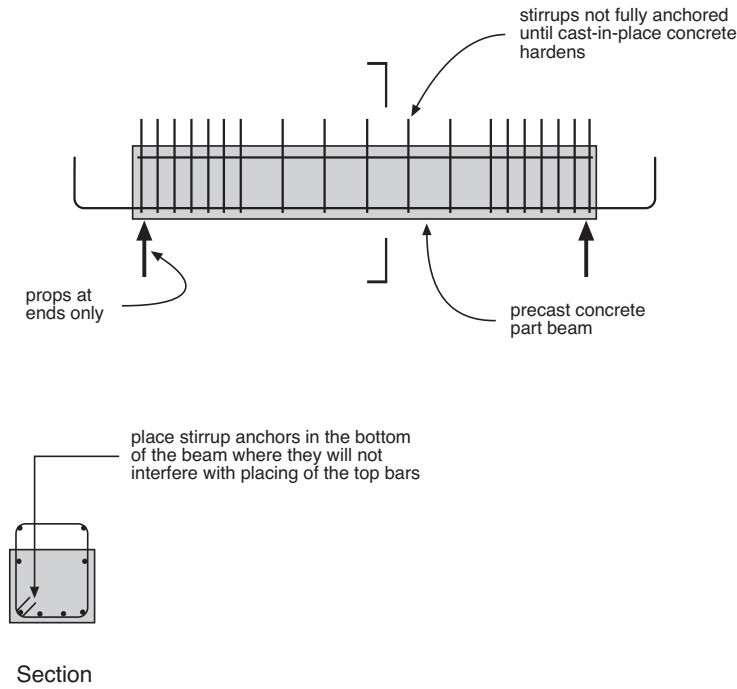


Figure B1.2: Uncertain anchorage of stirrups providing shear strength of a precast beam during placement of cast-in-place concrete

Appendix B2: Experimental Tests on Precast Concrete Frame Midspan Joint Connections for System 2

Tests on midspan joints of the System 2 precast concrete beam-column configuration have been conducted at Works Central Laboratories [B2.1] and the University of Canterbury [B2.2, B2.3].

The Central Laboratories' tests joint details and test rig are shown in Figure B2.1. The test specimens were H-shaped. During cyclic loading there was no apparent degradation in the three cast-in-place midspan joints. In these tests, however, the test frame applied lateral restraint to beam ends and hence restrained beam growth caused by plastic hinging at the beam ends. The resulting axial compression in the beams would have caused a more favourable performance of the midspan joints than had there been no axial load.

The University of Canterbury tests involved four H-shaped test specimens that were subjected to cyclic horizontal loading at the column tops with the column bases pinned (see Figure B2.2(a)). The base support of one column was also on a sliding bearing, thus avoiding as far as possible the presence of axial forces in the beam due to beam growth during testing. Figure B2.2(b) shows the details of the four test specimens.

Unit 1 had a midspan connection with hooked laps which performed extremely well during testing. At high ductility factors there was only very minor cracking, namely one diagonal tension crack and some very fine shrinkage cracks. Figure B2.2(c) shows the condition of Unit 1 after completion of the test. The undamaged midspan region contrasts with the plastic hinge damage. It is to be noted that once diagonal tension cracking commences in the beam, the "tension shift" effect means that longitudinal reinforcement at the midspan connection will have substantial tensile stresses, although the bending moment diagram for this test loading has zero bending moment at midspan. The maximum stress measured in the longitudinal beam steel at midspan in Unit 1 was 42% of the yield strength. Unit 2 incorporated the double-hooked "drop in" splice bars shown in Figure B2.2(b). Once again, the performance of the connection during testing was excellent. Unit 3 had a midspan connection with conventional straight bar splices and performed extremely well during testing.

These tests on Units 1, 2 and 3 demonstrated that a length of beam of at least one effective beam depth d at each end adjacent to the column face is sufficient to

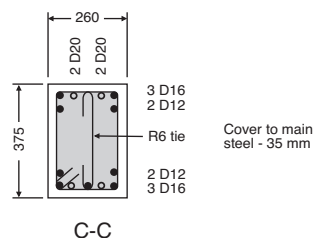
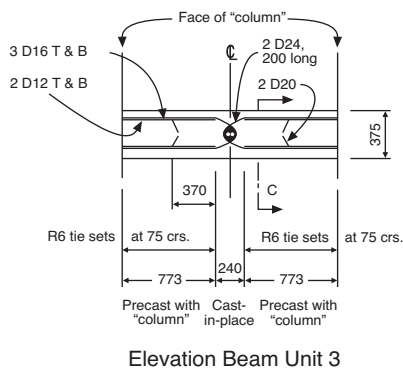
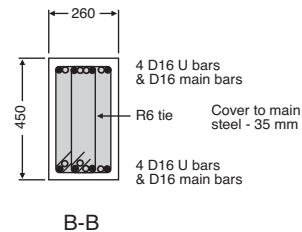
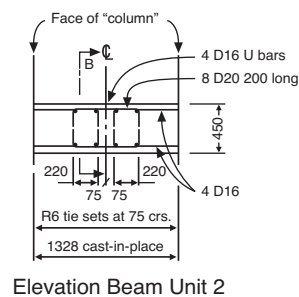
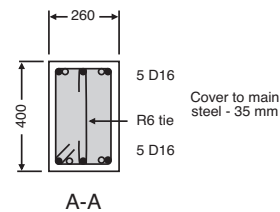
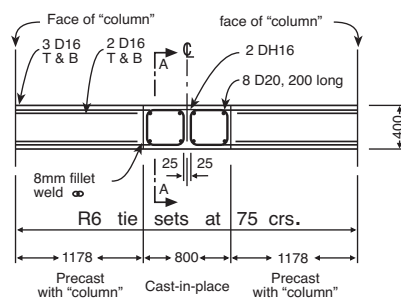
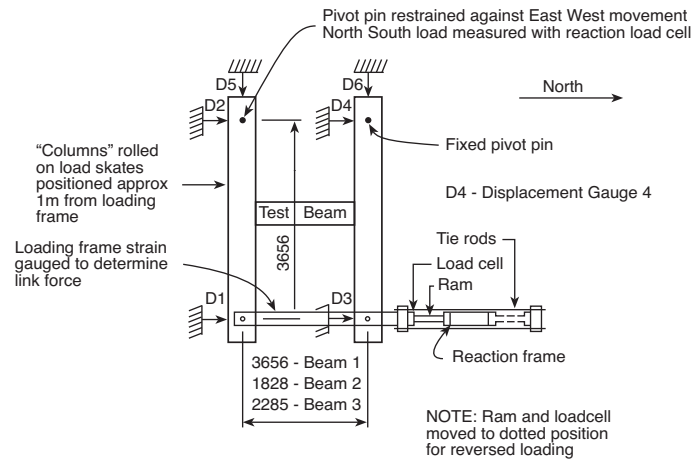
achieve adequate plastic hinge behaviour as required in ductile frames. Therefore, the provision in the 1995 concrete design standard that allows splices to commence at a distance d from the column face is justified. This design provision gives designers considerable flexibility when detailing midspan connections.

The performance of Unit 4 was less ductile than that of Units 1, 2 and 3. For Unit 4 there was a drop in lateral load carrying capacity at a drift of 1.5% when the displacement ductility factor was 4. It was observed that crushing of the concrete had occurred at the inside of the bends of the diagonal bars of Unit 4 due to radial pressure there from the diagonal bars in tension and the tie force from the diagonal bars in compression. When this crushing occurred in the test it meant that the truss mechanism of Fig. B2.3, which was used in the design of the beam, could not develop effectively. It is evident that caution is required when designing the detail shown for Unit 4 [B2.3].

An improved method for designing Unit 4 would be to provide transverse bars at the inside of the bends of the diagonal bars to improve the bearing strength there. Also, ties are required around the ends of the bars terminated at the ends of the strong region to permit proper transfer of forces. Reference B2.3 gives a recommended design procedure.

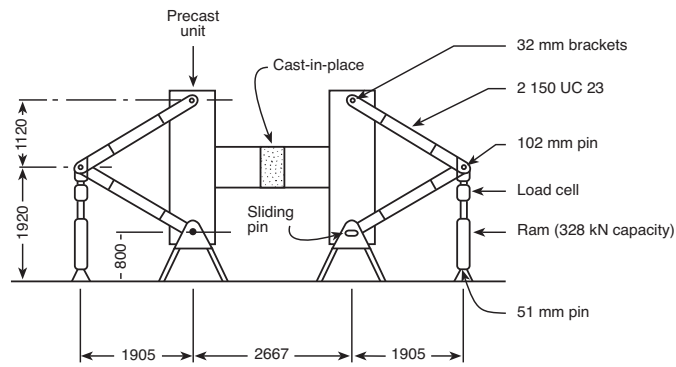
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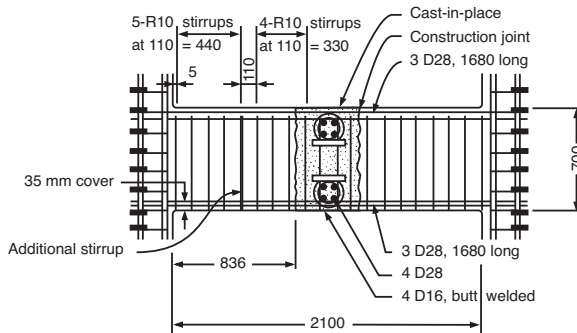


(b) Details of midspan test specimens

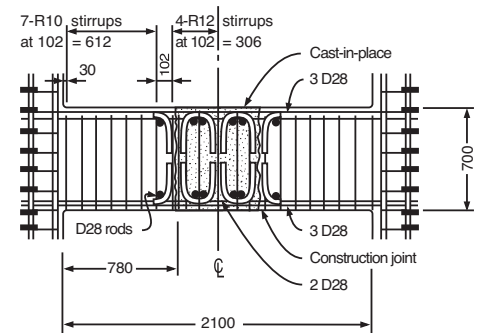
Figure B2.1: Details of the Works Central Laboratories loading rig and midspan joint test specimens [B2.1]



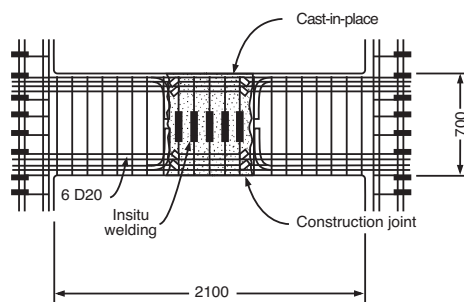
(a) Loading rig



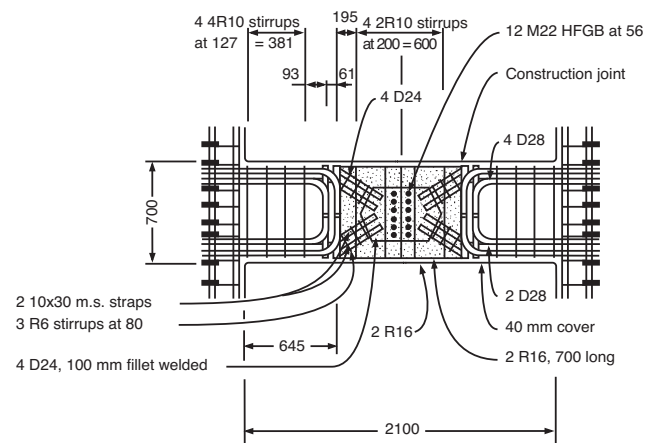
Unit 1 - Overlapping 180° hooks



Unit 2 - Double 90° hooked drop in bars



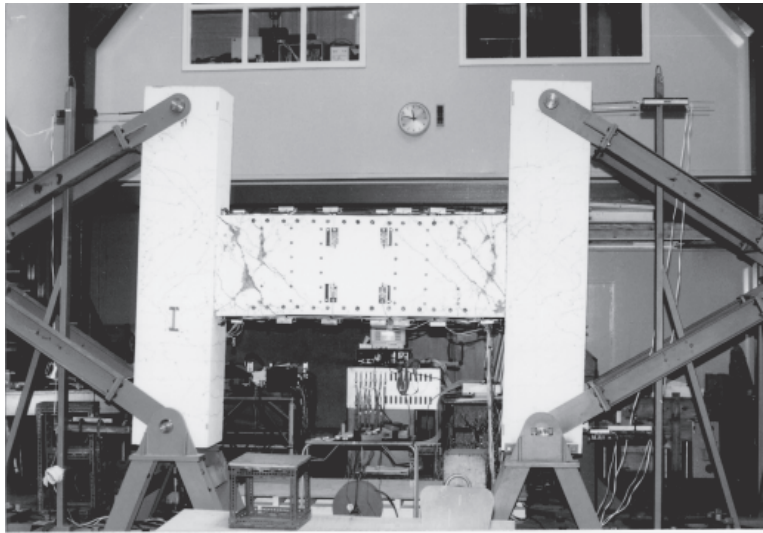
Unit 3 - non-contact straight lap splices



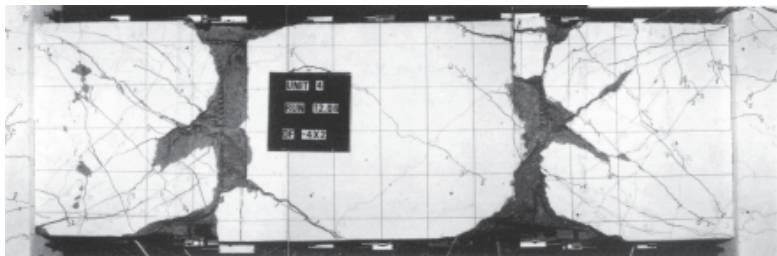
Unit 4 - Strong end regions and diagonal bars welded to bolted plates

(b) Details of midspan test specimens

Figure B2.2: Details of the University of Canterbury loading rig and midspan joint test specimens [B2.2]



(c) Unit 1 at end of test



(d) Beam of unit 4 at end of test

Figure B2.2 (continued): Details of the University of Canterbury loading rig and midspan joint test specimens [B2.2]

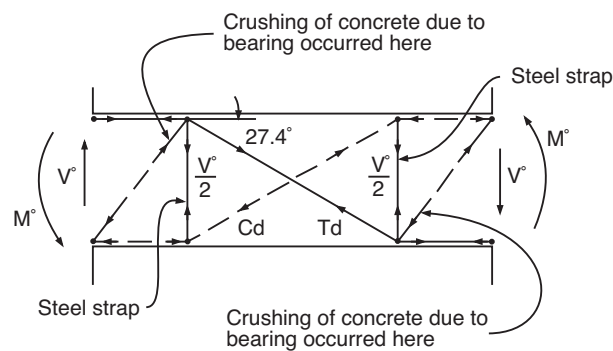


Figure B2.3: Truss model assumed for design of Unit 4 [B2.3]

Appendix B3: Laboratory Tests on the Performance of Grouted Connections for System 2

Test of Components

Tests were carried out at Works Central Laboratories [B3.1, B3.2, B3.3] and at the University of Canterbury [B3.4] to investigate the behaviour of grouted column bars. The purposes and the conclusions of these tests were as follows.

- (i) To measure the bond strength of the main column steel within grouted sleeves, and to compare the results with the bond strength of bars in cast-in-place construction.

The test results indicated that grouted bars had a slightly improved pull-out strength when compared with bars in cast-in-place construction. The improved performance was attributed to the strength of the grout being greater than that of the concrete. Failure mechanism did not involve the bond between the bars and grout, but occurred due to breakdown of bond between the precast concrete and ducting. For both the grouted and cast-in-place bars, very high bond stresses were developed, with some bars reaching yield strength even though the development length was well below recommended code values.

- (ii) To assess the effectiveness of column ties passing around grout sleeves.

Monotonic and cyclic pull-out tests were performed on R12 ties which passed around grouted ducts containing longitudinal bars and around longitudinal bars in cast-in-place construction. In all cases the R12 ties yielded. There was little difference in the stiffnesses measured in cyclic and monotonic loading tests. The R12 tie was slightly stiffer when bent around bars in cast-in-place construction than when bent around grouted ducts containing bars. This detail, therefore, involving the use of grouted corrugated metal tubing can be used with confidence. In these tests the compressive strength of the grout was 51 MPa and the concrete, 35 MPa. Until further laboratory test results are available, as is recommended in Chapter 6, the grout should be at least 10 MPa stronger than the specified compressive strength of the precast concrete.

- (iii) To inspect the completeness of the grout fill of sleeves.

The tests involved pumping grout into the horizontal joint between the beam soffit and column top, and up through the vertical ducts in the beams. Cementitious grouts with an expanding agent were used. Inspection of the grout, after saw cuts had been made, confirmed that grout had completely filled the horizontal joints and vertical ducting. A number of recommendations were made regarding the grout design and grouting operation which are summarized in Chapter 6.

- (iv) To evaluate the use of a performance test unit, envisaged as a specimen which is clamped on to the structure during the grouting operation.

Concern has been expressed about the difficulty of assessing the quality of grouting after the operation has been completed. This test indicated that a grout specimen which is clamped on to the structure and later removed for strength testing, was not practical. This means that greater emphasis must be placed on inspection *during* the grouting procedure.

Test of Beam-column Joints

Full-scale tests on two System 2 moment-resisting joints were performed at Works Central Laboratories [B3.2, B3.3] and at the University of Canterbury [B3.4]. As is typical for a System 2 joint, longitudinal column reinforcing bars passed up through corrugated metal tubes located in the precast beam. A proprietary cementitious grout was used. The grouting operation is shown in progress in Figure B3.1. The beam-column joints were subjected to cyclic loading to investigate their stiffness and strength degradation. Test results showed that behaviour of these joints was similar to that of a conventional cast-in-place beam-column joint.

There was no indication of bond failure between the column bars and the precast unit. The overall joint performance was satisfactory up to beam deflection levels of displacement ductility factor 10 for the Works Central Laboratories test, and to at least 6 for the University of Canterbury test, at which stage the damage at the beam plastic hinge was sufficient to cause a reduction in strength. These results indicate that System 2 provides a very satisfactory beam-column joint providing that the design and workmanship are adequate.



Figure B3.1: Grout being pumped into the ducts

References

- B3.1 Thurston, S J. *Precast Grouted Beam-Column Joint*, Report S-86/9, Ministry of Works and Development Central Laboratories, Lower Hutt, 1986, 49 pp.
- B3.2 Beattie, G J. "Recent testing of precast structural components at Central Laboratories", *Proceedings of the New Zealand Concrete Society Conference*, Wairakei, October 1989, pp. 111-118.
- B3.3 Stevenson, R B and Beattie, G J. *Cyclic Load Testing of a Beam-Column Cruciform Incorporating a Precast Joint Zone and Column Bars Grouted in Drossback Ducts*, Report 89-B5204/2, Central Laboratories, Works and Development Service Corporation (NZ) Ltd, Lower Hutt, 56 pp.
- B3.4 Restrepo, J I, Park, R and Buchanan, A H. "Tests on connections of earthquake resisting precast reinforced concrete perimeter frames of buildings", *Journal of the Prestressed/Precast Concrete Institute*, Vol. 40, No. 4, July-August 1995, pp 44-61.

Appendix C1: Assessing the Influence of Thermal Gradients, Creep and Shrinkage Strains in Composite Concrete Members

The sun acting on a concrete surface, in low wind conditions, can generate a significant thermal gradient in the member. Measurements made on bridges indicate that temperatures on deck surfaces may be expected to rise by 25°C to 30°C above ambient one or two times a year. Similar conditions occur on concrete roofs and exposed suspended slabs. The deflections and support rotations that develop as results of these actions have in the past caused appreciable damage (see Section 2.3.4). Thermal gradients used for bridge design may be used to assess the appropriate temperature profiles for roofs and other elements exposed to the sun, see reference C1. A method of assessing the structural actions due to thermal gradients is outlined later in this appendix.

Creep, shrinkage in precast prestressed concrete members can result in significant deflections in certain situations. This can occur in two ways:

- If the unit is made up of different thicknesses, the thinner portions generally shrink more than thicker elements causing distortion to occur.
- Reinforcement, normal or prestressed, restrains shrinkage movements. If this is asymmetrical in the section, the restraint leads to curvature.

When concrete topping is placed on precast units additional deformation generally occurs due to differential shrinkage between the precast and cast-in-place concrete. In addition, some redistribution of dead load and prestress can occur between the precast member and the composite section with creep of the concrete.

Where long spans are used, or the deflection of the floor is important, a number of different methods of analysis are available to predict the deflected shape. The simplest of these, which is the modified modulus method [C.2 and C.3], is described in the later part of this appendix. A number of other different approaches are described in references C.4 and C.5.

With any method of analysis the concrete properties have to be known. Tests on New Zealand concrete [C.5] have indicated that the creep values can be predicted with reasonable accuracy from the CEB-FIP recommendations [C.6], but the shrinkage values in

this document are on the low side. The free shrinkage strain and creep coefficient for a typical 65 mm thick 25 MPa concrete topping are of the order of 650×10^{-6} and 3.5 respectively. For typical precast concrete units with a maturity of about 3.5 weeks the free shrinkage strain and creep coefficient are reduced to about half the values for the cast-in-place topping concrete. Precast units such as “Dycore” (hollowcore), which are made from low water content concrete, have a free shrinkage of the order of 350×10^{-6} and a creep coefficient of about 1.8 at the end of its initial curing period (two days). These reduce respectively to about 100×10^{-6} and 1.2 after a further period of three weeks. It should be noted that the basic creep and shrinkage values cannot be predicted accurately without testing and appreciable variation in these values may be expected with different aggregate types, cement contents and curing conditions. The thermal coefficient of concrete is also subject to variation with aggregate type and mix constituents, but a typical value is 12×10^{-6} per $^{\circ}\text{C}$.

Analysis

The methods of analysis for thermal, creep and shrinkage induced actions in concrete members all follow very similar steps, as outlined below.

- 1 Find the elastic (or equivalent elastic) transformed section properties of the members.
 - For thermal gradients and other situations where the induced stresses are not sustained for long periods of time, section properties based on elastic transformed sections are found. Allowance should be made for differences in the modulus of elasticity, E_c , of insitu (topping) and precast concretes. Transformed properties should generally include both the passive and prestressed reinforcement.
 - Where actions arise from shrinkage of the concrete allowance must be made for the relaxation in stresses due to creep. For shrinkage, which develops gradually over a long period of time, the effective modulus of the concrete(s) should be taken as $E_c/(1+0.6\phi)$, where ϕ is the creep factor for the concrete and E_c is the initial elastic modulus.

- For these cases where loads are sustained for long periods of time, such as dead load or prestress, transformed properties are determined using an effective elastic modulus of the concrete(s) of $E_c/(1+\phi)$.
- 2 Butresses are assumed to hold the structure so that no strain can develop in the member. The stresses in the concrete are found and the forces that the buttresses apply to the members are determined. There is no deflection or deformation of the structure associated with this step.
 - For the differential temperature case the stresses in the concrete in the buttress held condition are equal to $(T \times \alpha \times E_c)$, where T is the temperature rise of the fibre being considered and α is the coefficient of thermal expansion of the concrete.
 - For the differential shrinkage case the buttress held stress in the concrete are equal to the value $S_d E_c/(1 + 0.6\phi)$, where S_d is the differential shrinkage of the concrete assuming this value was free to develop.
 - 3 Forces of equal but opposite sign to the buttresses forces found in step 2 are applied to the structure to cancel out the buttress forces. The effective elastic moduli defined in step 1 must be used. The deflection is associated with this step.
 - 4 The resultant stresses and deflections are now found by adding the stresses and deflections found in steps 2 and 3.

If the form of a structure is changed after some load is applied redistribution occurs as the concrete creeps.

This situation arises, for example, when insitu concrete is added to a precast unit. The dead load of the insitu concrete and the self weight of the unit are initially resisted by the precast unit alone. However, as creep develops some of this load is redistributed to the composite section. A method of assessing the resultant deflection using the modified effective modulus approach is given in references C.2 and C.3.

References

- C.1 Transit New Zealand. *Bridge Manual: Design and Evaluation*, Wellington, New Zealand, 1991 draft, p 131.
- C.2 Sritharan, S and Fenwick, R C. "The influence of creep and shrinkage on the behaviour of prestressed concrete beams", *Proceedings of NZ Concrete Society Technical Conference*, 1989, pp 17-25.
- C.3 Sritharan, S and Fenwick, R C. "Creep and shrinkage effects in prestressed beams", *Magazine of Concrete Research*, Vol. 74, No. 170, March 1995, pp 45-55.
- C.4 American Concrete Institute. *Designing for Creep and Shrinkage in Concrete Structures*, ACI Special Publication SP76, American Concrete Institute, 1982, pp 484.
- C.5 Bryant, A H , Wood, J A and Fenwick, R C. "Creep and shrinkage in concrete bridges", *RRU Bulletin 70*, National Roads Board, 1984, 106pp.
- C.6 Comité Euro-international du Béton. *CEB-FIP Model Code 1990*, Thomas Telford, London, 1993, 437pp.

Appendix C2: Method for Locating the Position of Creep and Shrinkage Cracks and Assessing Crack Widths

In a precast concrete floor with cast-in-place topping it is possible to estimate over what length of floor the formation of a crack relieves volume change strains and from this information the maximum possible crack width can be assessed. In the example shown in Figure C1(a) the crack causes the stresses in the shaded area to be reduced. The maximum crack width c , at a point such as C, is approximately equal to the volume change strain $\times L/2$, where L is the length over which the stresses are reduced. The formation of the wide crack at B could have been avoided if a concentrated band of reinforcement had been run from the wall DB into the topping concrete in the slab, as shown in Figure C1(b). With this steel, provided a sufficient quantity is used, the crack width is controlled at B, and the length over

which the stresses are relieved when the crack forms from A to B is greatly reduced. As illustrated, further cracks may now be expected to open up and absorb some of the shrinkage and thermal strains. The 30° angle dividing the restrained and unrestrained floor regions due to the presence of a wall, or some other relatively rigid element, is a simple working approximation that is based on the results of a series of finite element analyses [C.5].

Reference

C.5 Bryant, A H , Wood, J A and Fenwick, R C.
“Creep and shrinkage in concrete bridges”,
RRU Bulletin 70, National Roads Board,
1984, 106pp.

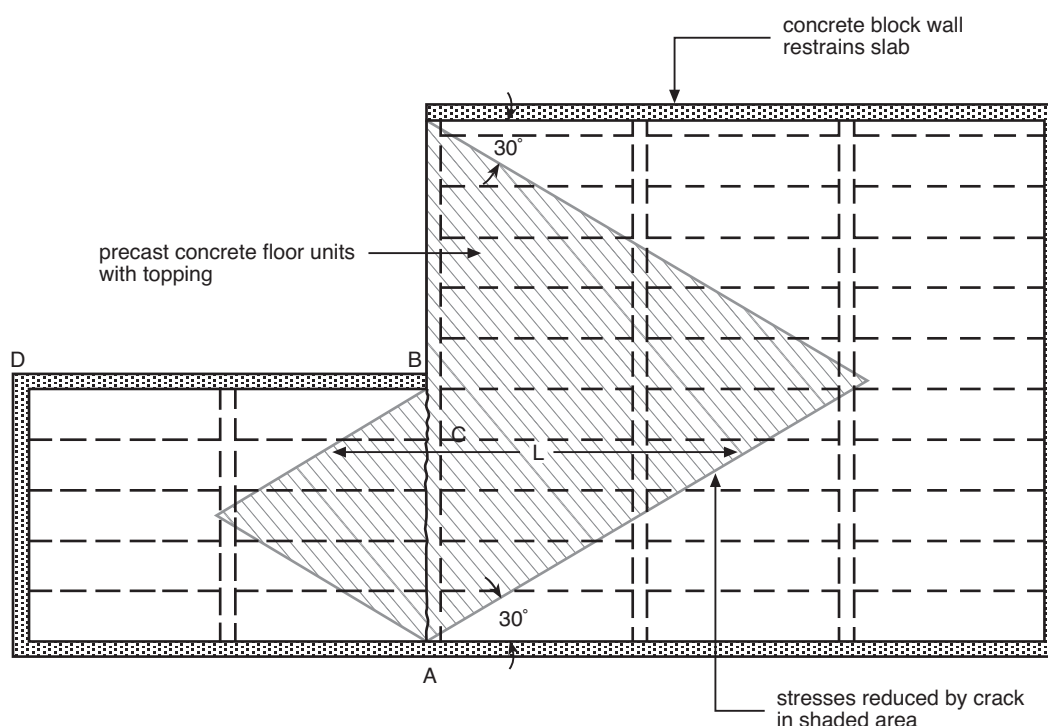


Figure C1a: Examples of crack locations in slabs - crack in a floor slab partially restrained by block walls

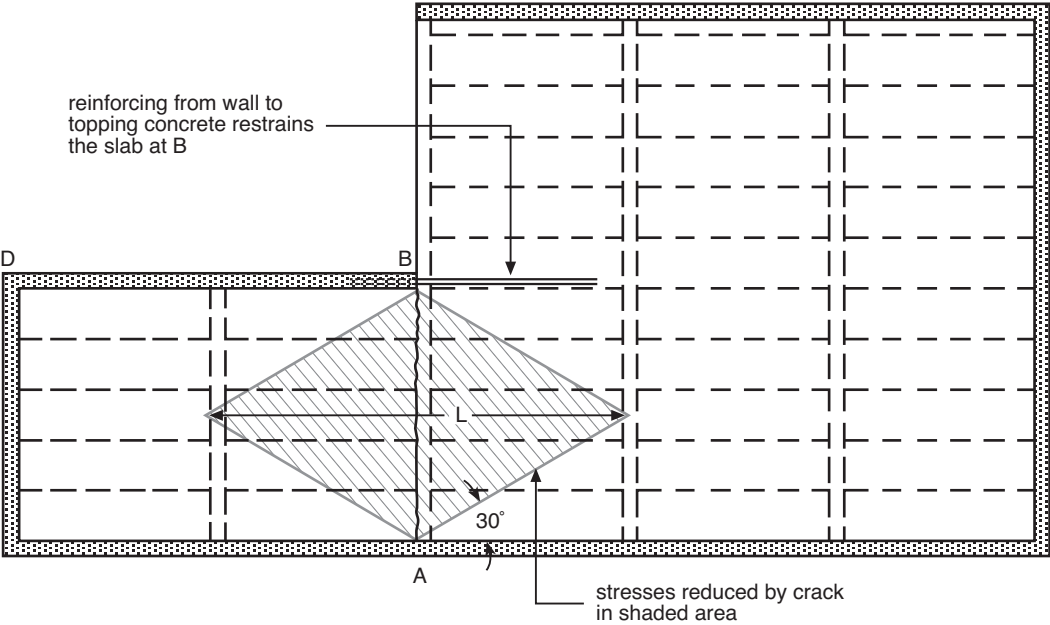


Figure C1b: Examples of crack locations in slabs — an additional band of reinforced cement helps to control the crack widths

Appendix D1: Criteria for the Selection of Fixings

QUESTION	ACTIVITY	DETAILS TO BE CONSIDERED
What loads are to be carried by the fixing assembly?	Nature of loading has been ascertained previously by the designer	<div>Dead</div> <div>Sustained</div> <div>Tension</div> <div>Live</div> <div>Intermittent</div> <div>Compression</div> <div>Temporary</div> <div>Cyclic</div> <div>Shear</div> <div>(Construction)</div> <div>Dynamic</div> <div>Torsion</div>
What kind of fixings are available for this application?	Description of fixing types	<div>Cast-in fixings</div> <div>Expanding fixings</div> <div>Bonded fixings</div>
What are their performance characteristics?	Test data	<div>Short-term tensile loading</div> <div>Sustained axial loading</div> <div>Cyclic loadings, shear loading</div> <div>Load/deformation characteristics, e.g. ductility</div> <div>Elevated and low temperatures, resin capsule fixings</div> <div>Behaviour and vulnerability under fire conditions</div>
Are these all the factors that influence performance?	Limitations of use of particular types of fixing related to base material characteristics	<div>Nature of base material</div> <div>Bonded fixings</div> <div>Sensitivity to tolerance in fabrication, and distortion.</div>
What about corrosion?	Durability	<div>Types of corrosion</div> <div>Corrosion prevention</div>
What has to be considered in the structural design of a fixing assembly?	Design criteria	<div>Principles</div> <div>Safety margins</div> <div>Spacing of fixings</div> <div>Embedment depth in base material</div>
How does the designer convey his requirement to the user?	Specification and drawings	<div>Who is responsible for doing what?</div> <div>Consult with supplier and user</div> <div>Specification</div>
Anything else?	Activities not covered above	<div>Consider the advantage or disadvantages of casting-in (in concrete) or using a fixing in a drilled hole</div> <div>Is the fixture permanently attached or will it be removable? This may influence the detail of the connection (e.g. accessibility)</div> <div>Consider method of attachment direct by bolting or indirect by use of bracket or corbels which themselves may be secured with a fixing</div> <div>Allow for possible differential movement between the base material and the fixture</div> <div>Appearance: hidden or seen. Staining of fixture or base material</div> <div>Total cost: materials plus labour</div>

Table D1: Checklist for selection of fixings

Appendix D2: Embedments Subject to Corrosion

The following clauses indicate areas of concern that should be addressed by designers. The clauses are from PW 81/10/1:1985, *Guidelines for the Seismic Design of Public Buildings*, Civil Engineering Directorate Publication, Ministry of Works and Development. This publication is no longer operational as a standard citing document, but the technical content provides sound engineering advice in this area. Works and Development Services Corporation (NZ) Ltd's permission to publish these clauses is acknowledged.

12.4.6.2 Fixings exposed to weather or corrosive conditions

Fixings exposed to weather or corrosive conditions, shall be of stainless steel suitable for the particular application except that where they are visible and can be inspected and maintained they may be galvanised and painted mild steel.

In the case of stainless steel, care shall be taken to avoid creating narrow crevices which may entrap water. If present these are to be filled with a chromated mastic (see Clause 12.4.7.2).

Comment: Metal fixings exposed to weather create corrosion and maintenance problems. Whenever possible the building facade should be designed in such a way that such fixings are not required. Choice between galvanised mild steel and stainless steel should take account of likely maintenance costs including scaffolding, etc., required to gain access to them. Galvanised steel exposed to weather has a limited life and should always be painted. Repainting intervals will depend on exposure conditions.

12.4.6.3 Dissimilar metals

Corrosion may result through the galvanic action of dissimilar metals in contact. Stainless steel and mild steel should not be welded together.

Comment: Welding together of mild and stainless steels is technically feasible but can introduce embrittlement and corrosion problems.

12.4.6.4 Stainless steel types

The following stainless steel types shall be used:

AISI 304 (304 L or 321 if to be welded): General purposes.

AISI 316 (361 L if to be welded): For corrosive conditions or marine atmosphere salt laden air.

Comment: Special applications may call for other types and expert advice should be obtained in these cases. Stainless Steel Alloys Ltd, Box 31193, Lower Hutt, publish useful references.

12.4.7.2 Stainless steel and bolts

The threaded portion of stainless steel bolts shall be coated with an approved chromated mastic or Dulux Stag A red jointing paste to prevent corrosion due to oxygen exclusion. Nuts are not available in type 316 alloy, and heads of type 316 bolts shall therefore be exposed where corrosive conditions exist.

Comment: Chromates are among the most effective corrosion inhibitors, but if used in adequate concentration they then act as a corrosion accelerant. The manufacturer's instructions should be followed.

12.4.8 Admixtures containing chlorides

Admixtures containing chloride ions shall not be used in grouts, mortars or concrete in contact with fixings.

Comment: Admixtures containing chloride ions may have a corrosive effect on fixings and other embedded metals. Also, if dissimilar metals are present, chlorides may accelerate any galvanic action.

12.4.9 Cold worked steel

Cold worked steel which is to be galvanised or has the cold worked area subsequently heated by welding, shall be heat treated unless test evidence shows that embrittlement will not occur.

Comment: Galvanising of cold worked steel can result in serious embrittlement in the cold worked areas. Refer to ASTM A123 and A143 for guidance. There is also a serious embrittlement risk when steel is cold worked and subsequently heated by adjacent welding.

Strain ageing embrittlement is accelerated by higher temperatures such as those encountered during galvanising and welding, and results in

loss of ductility and even in brittle fracture at bends during the ordinary process of handling and erection.

Appendix E1: Example Calculation for Embedded Steel Connectors

NOTE: Refer to Chapter 7 “Embedded Steel Connectors” for background theory to these examples.

E1.1 Example 1 — Weld Plate with Studs

Design a weld plate with studs to be cast into a 400 mm square reinforced concrete column. The concrete compressive strength of the column is $f'_c = 30$ MPa. A 360 UB is to be fixed to the weld plate. The column hoops are widely spaced and cannot be relied on for transferring the forces in the connection. The following design actions have been derived from the structural analysis:

$$V^* = 100 \text{ kN}$$

$$M^* = 24 \text{ kNm}$$

$$N^* = 20 \text{ kN (tension)}$$

Figure E1.1 shows an elevation of the weld plate connection detail. Tension and compression forces are found in equilibrium. The shear force, V_b , transferred by the bearing of the plate on to the concrete will be ignored. Hence, the shear force needs to be transferred by friction at the compressive block of stresses, V_f , and

by direct shear through the top and bottom studs, V_{dt} and V_{db} , respectively.

1 Design of the Steel Embedment

1.1 Design for Shear

Ensure that the dependable shear strength of the connector is equal or greater than the design shear force

$$\phi V_n \geq V^* \quad (\text{E1.1})$$

where the strength reduction factor for a steel connector is $\phi = 0.85$. Try using two rows of 2-12.7 mm diameter Nelson studs (see Figure E1.1).

The nominal shear strength, V_n , is the sum of the shear force transferred by friction, V_f , plus the force transferred by direct shear through the top and bottom studs, V_{dt} and V_{db} , respectively.

$$V_n = V_f + V_{dt} + V_{db} \quad (\text{E1.2})$$

The shear force transferred by friction is computed from a shear friction coefficient $\mu_f = 0.7$ recommended by the New Zealand Concrete Structures Standard, NZS 3101:1995 [E1]. Thus

$$V_f = 0.7 \times 58.6 = 41.0 \text{ kN}$$

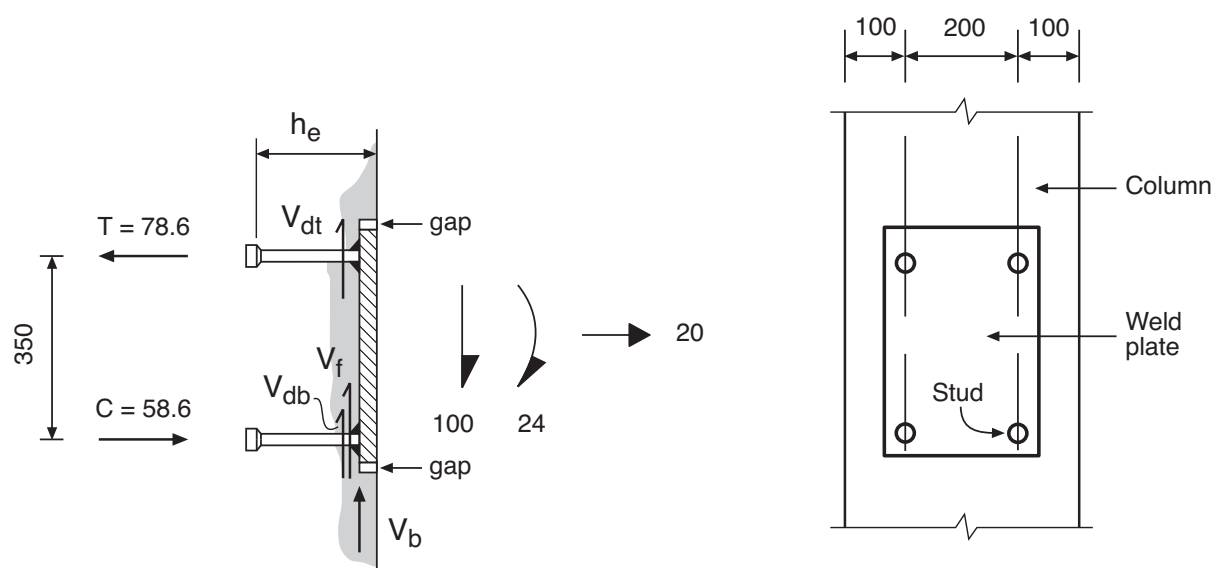


Figure E1.1: Elevation of weld plate connection and design actions

The force transferred by direct shear through the bottom studs is computed ignoring the presence of axial force. The ultimate shear strength under pure shear stress conditions is equal to $0.6 f_{ut}$. Therefore, the shear force transferred by these studs is

$$V_{db} = 0.6 f_{ut} A_{sb}$$

Where A_{sb} is the cross-section area of the bottom studs.

Assume two bottom studs. From Figure E1.2 the shear force carried by the bottom studs is

$$V_{db} = 0.6 \times 415 \times 2 \times 127 \times 10^{-3} = 63.2 \text{ kN}$$

The force required to be carried by direct shear through the top studs is found by substituting Equation E1.2 in to Equation E1.1 and rearranging for V_{dt}

$$\begin{aligned} V_{dt} &= \frac{V^*}{\phi} - (V_f + V_{db}) \\ &= \frac{100}{0.85} - (41.0 + 63.2) = 13.4 \text{ kN} \end{aligned}$$

The required area of studs to transfer V_{dt} is

$$A_{st}^v = \frac{13.4 \times 10^3}{0.6 \times 415} = 54 \text{ mm}^2$$

1.2 Design for Flexure

The nominal tensile strength of the studs, T_n , is given by

$$T_n = A_{st} f_{ut}$$

where A_{st} is the cross-section area of the studs and f_{ut} is the ultimate tensile strength of the stud when a well-defined yield plateau is not apparent.

Now ensure that the dependable tensile strength of the connector is equal to or greater than the design action

$$\phi T_n \geq T \quad (\text{E1.3})$$

The area of top studs required for transferring tension only, A_t , is from Equation E1.3

$$A_{st}^t = \frac{T^*}{\phi f_{ut}} = \frac{78.6 \times 10^3}{0.85} \frac{1}{415} = 222.8 \text{ mm}^2$$

1.3 Combined Flexure and Shear

The top studs are required for transferring tension force, resulting from flexure and axial tension, and shear force simultaneously.

The area of studs, A_{st} , required for transferring the combined action is

$$\begin{aligned} A_{st} &= \left[(A_{st}^v)^2 + (A_{st}^t)^2 \right]^{1/2} \\ &= [54^2 + 222.8^2]^{1/2} = 229 \text{ mm}^2 \end{aligned}$$

The area provided by 2-12.7 mm diameter studs is $2 \times 127 = 254 \text{ mm}^2$, which is greater than 225 mm^2 . Hence the design is satisfactory.

2 Embedment Length

Use capacity design to ensure that the dependable strength of the concrete resisting mechanism, ϕT_c , is equal to or greater than the probable strength of the studs, λT_s

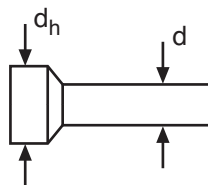
$$\phi T_c \geq \lambda T_s \quad (\text{E1.4})$$

where

- ϕ = strength reduction factor for concrete in flexure and axial force
- T_c = 5 percentile lower characteristic strength of the concrete resisting mechanism
- λ = ratio between probable and 5 percentile lower characteristic tensile strength for studs (no yield plateau)
- T_s = tension force in the studs, based on the 5 percentile lower characteristic tensile strength of a stud.

The Ψ -method given in references E2 and E3 recommend for T_c

$$T_c = 10.7 \xi_R h_e^{3/2} \sqrt{f'_c} \quad (\text{N})$$



d (mm)	d _h (mm)	A _s (mm ²)
12.7	25	127
15.9	32	198
19.1	38	285

Figure E1.2: Properties of Nelson studs

where ξ_R is a factor that considers edge, concrete cracking and group effects, and h_e is the embedment length at the stud, measured as shown in Figure E1.1.

In lieu of data supplied by the manufacturer, use $\lambda = 1.21$ as recommended in Reference E3. Thus

$$0.85 \times 10.7 \xi_R h_e^{3/2} \sqrt{30} \geq 1.21 \times 127 \times 415$$

$$h_e \geq \frac{118}{(\xi_R)^{2/3}} \text{ mm} \quad (\text{E1.5})$$

Evaluation of factor ξ_R [E3]

$$\xi_R = \Psi_{CR} \Psi_{CX} \Psi_{SX} \Psi_{CY} \Psi_{SY} \quad (\text{E1.6})$$

For this example and with reference to Figures E1.1 and E1.3

$$\Psi_{CR} = \Psi_{SY} = \Psi_{CX} = 1$$

Group effect factor Ψ_{SX}

$$\begin{aligned} n_x &= 2 \\ s_{cr} &= 3h_e \\ s &= 200 \text{ mm} \end{aligned}$$

assume $h_e = 260$ mm and then check. Hence $s_{cr} = 780$ mm.

$$\Psi_{SX} = \{1 + (2 - 1) \times 200 / 780\} / 2 = 0.63 \leq 1$$

Edge effect factor, Ψ_{cy}

$$\begin{aligned} c_x &= 100 \text{ mm} \\ \Psi_{CY} &= \{0.3 + (0.7 \times 100 / 390)\} = 0.48 \leq 1 \end{aligned}$$

Substituting Ψ_{CR} , Ψ_{sy} , Ψ_{cx} , Ψ_{sx} and Ψ_{cy} in Equation E1.6 results for ξ_R

$$\xi_R = 1 \times 1 \times 0.63 \times 0.48 \times 1 = 0.30$$

Now substituting ξ_R in Equation E1.5

$$h_e \geq 118 / (0.30)^{2/3} = 261 \text{ mm}$$

Note that the value initially assumed for h_e is very close to the value obtained. Consequently, there is no need to carry a second iteration.

E1.2 Example 2 — Cast-in RHS Stud

Design the cast-in RHS stud for the force shown in Figure E2.1.

$$\begin{aligned} 102 \times 76 \times 6.3 \text{ RHS} \quad f_y &= 225 \text{ MPa} \\ f'_c &= 35 \text{ MPa} \end{aligned}$$

A cast-in RHS stud is to support a load of

$$V^* = 100 \text{ kN} \quad (\text{E1.7})$$

Use the Steel Structures Standard [E17]. Check RHS for shear,

$$\begin{aligned} V_n &= 0.55 A_w F_{yw} \\ &= 0.55 \times 6.3 \times 102 \times 2 \times 225 \\ &= 159 \text{ kN} \end{aligned}$$

$$\phi V_n = 135 \text{ kN} > V^*, \text{ satisfactory}$$

Check RHS for flexure

$$\begin{aligned} M_n &= S F_y \\ &= 64 \times 225 \\ &= 14.4 \text{ kNm} \end{aligned}$$

$$M^* = 100 \times 0.1 = 10 \text{ kNm}$$

$$\phi M_n = 12.2 \text{ kNm} > 10 \text{ kNm}, \text{ satisfactory}$$

The dependable bearing stress f_b on the concrete is

$$0.85 \phi f'_c = 19.3 \text{ MPa}$$

where $\phi = 0.65$ (NZS 3101, Clause 3.4.2.2(f)) and $f'_c = 35 \text{ MPa}$

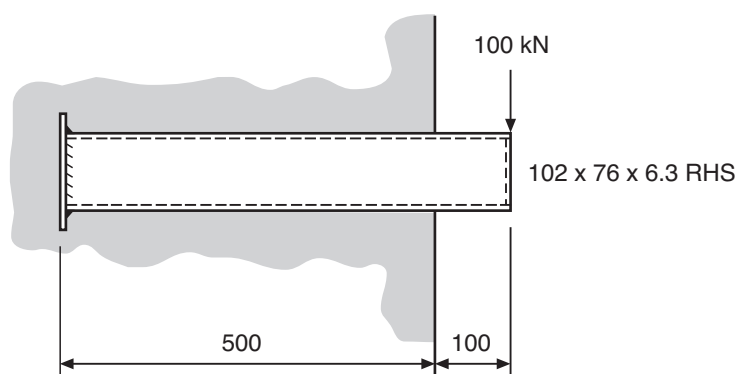


Figure E2.1: Elevation of cast-in RHS stud

The maximum bearing stress that the RHS can resist is limited by the flexural strength of the RHS wall spanning between the webs.

At collapse

$$M_p = \frac{wl^2}{16} \text{ (hogging and sagging plastic hinges)}$$

Dependable

$$M = SF_y = 225 \times \frac{1}{4} \times 1 \times 6.3^2 \\ = 2233 \text{ Nmm / mm}$$

$$\text{Maximum stress} = 2233 \times 16/70^2 = 7.3 \text{ MPa}$$

Hence allow 19.3 MPa under web and 7.3 MPa across flange of RHS, giving

$$12.6 \times 19.3 + (76 - 12.6) \times 7.3 = 706 \text{ N / mm}$$

The depth of stress blocks is 85% of the distance to the point of zero strain, and stress block lengths are a and b as shown in Figure E2.2)

$$V = -706a + 706b$$

$$M = 706a \times (1.176b + 0.676a) - 706 \frac{b^2}{2}$$

$$\text{For } V^* = 100 \text{ kN}$$

$$a = (b - 142.6) \text{ mm}$$

$$10 \times 10^6 = 706 (b - 142.6) \times$$

$$\left[1.176b + 0.676 (b - 142.6) \right] - 353b^2$$

giving

$$b = 268 \text{ mm}$$

$$a = 125 \text{ mm}$$

$$l = 462 \text{ mm (minimum), hence 500 mm embedment is OK.}$$

If, in addition, there was a tensile force in the RHS of 20 kN, an end cap plate should be welded to the end of the stud and transverse reinforcement should be proportioned to cross the potential failure cone and carry the tensile force.

References

E1 *Concrete Structures Standard, NZS 3101:1995*, Standards New Zealand, Wellington, 1995, 256 pp.

E2 *Fastenings to Concrete and Masonry Structures - State of the Art Report*, Comité Euro-International du Béton, Thomas Telford, 1994, 249 pp.

E3 Restrepo-Posada J I and Park, R. "Tensile capacity of steel connectors with short embedment lengths in concrete", *Research Report 93-6*, Department of Civil Engineering, University of Canterbury, August 1993, 51 pp.

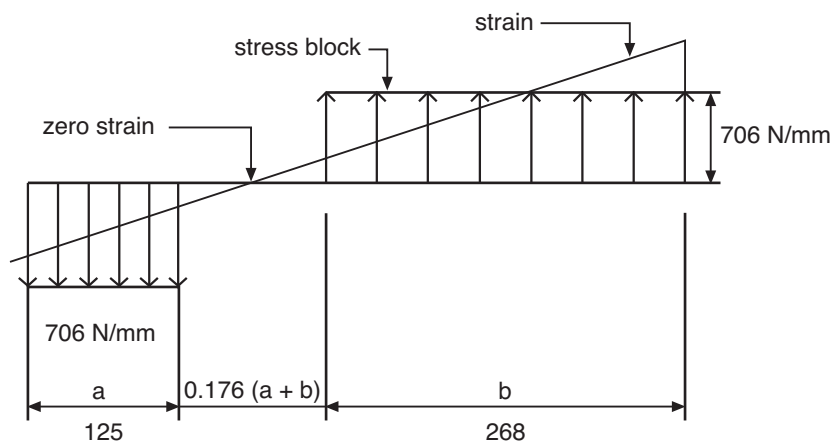


Figure E2.2: Equivalent load distribution in the embedded cast-in RHS stud

Details of Organisations Involved

New Zealand Society for Earthquake Engineering

The objects of the society, one of the earliest specialist groups of IPENZ, are to further those of the International Association for Earthquake Engineering as applicable to New Zealand, and to foster the advancement of the science and practice of earthquake engineering.

The Society has strong international affiliations, and has run a number of major international conferences: in 1965 the 3rd World Conference on Earthquake Engineering was held in New Zealand and the 12th World Conference on Earthquake Engineering is to be held in Auckland, New Zealand, from 30 January to 4 February 2000. The group has also been host to the South Pacific Regional Conference, and the Pacific Conferences of 1987, 1991 and jointly with the Australian Society, in 1995.

A principal activity is the maintenance of a pool of

experts in diverse disciplines able to travel at short notice to the sites of damaging earthquakes in countries comparable to New Zealand to learn first hand of the response of building structures and lifelines.

For further information please contact:

Andrew King (Chairman)
BRANZ
Private Bag 50-908
Porirua
phone 04-235-7604
e-mail: branzabk@branz.org.nz

Graham Quirke (Secretary)
PO Box 48-046
Upper Hutt
phone 04-528-4906
e-mail: quirke@ihug.co.nz

New Zealand Concrete Society

The New Zealand Concrete Society is a learned Society. The objectives of the Society are to “encourage a greater knowledge and understanding of all aspects of structural and architectural concrete and to support their development and use where appropriate”.

Membership of the Society encourages closer contact with many of the leading firms and individuals in the concrete industry — designers, manufacturers, constructors and material suppliers — to both give and take freely of ideas and experiences on a wide range of aspects of one of the most versatile and widely used building materials — concrete.

Additional membership benefits include entitlement to

the ‘New Zealand Concrete’ and ‘fib News’ publications. Reduced prices are available to members on publications, attendance at conference and most technical seminars and participation in Concrete and Prestressed Concrete Awards.

For further information please contact:

New Zealand Concrete Society
PO Box 12
Beachlands
Auckland
Phone: 09 536 5410, Fax: 09 536 5442
E-mail: toplev@xtra.co.nz

Centre for Advanced Engineering

CAE, the Centre for Advanced Engineering, was established in May 1987, as a centre for the promotion of innovation and excellence in engineering and technology, to commemorate the centenary of the School of Engineering at the University of Canterbury.

Vision Statement

To benefit New Zealand through promoting and encouraging the application of advanced engineering and technology.

Objective

CAE aims to enhance engineering knowledge within New Zealand by technology transfer and the application of New Zealand and overseas research to engineering-related issues of national importance.

Key Activities

- CAE undertakes major projects that bring together selected groups of practising and research engineers and other experts from industry, research organisations, local and central government, tertiary institutions, and the engineering profession.
- CAE also carries out smaller projects and organises seminars, workshops and conferences as opportunities arise.

CAE projects aim to:

- Be of national importance with wide public appeal and with tangible results.
- Facilitate technological co-operation amongst commercial and government organisations, tertiary institutions and the engineering profession.
- Identify deficiencies in New Zealand's technological capability and take action to promote the addressing of these deficiencies.
- Undertake technology transfer rather than original research.

Funding

CAE is an independent, non-profit organisation, financed mainly from the earnings of its trust fund. This fund, which currently stands at approximately \$2.3 million, consists of monies donated initially by 150 corporate donors and 750 individual donors during the 1987 Centennial Appeal, and more recently supplemented by further donations during the 10th Anniver-

sary Appeal. Other income is derived from sponsorship for specific projects, book sales and seminars. The University of Canterbury continues to make a major contribution to CAE by providing accommodation and financial services.

Administration

CAE is controlled by a Board of Directors comprising representatives from industry and commerce (including government and consulting engineers), the University of Canterbury and other tertiary educational institutions. The present Chairman is Dr Francis Small of Wellington. The Trust Fund is currently administered by the University of Canterbury under the direction of four trustees. CAE has two executive staff and three other staff engaged on publications and secretarial duties. Executive Director John Blakeley has overall responsibility for CAE activities and Projects Director John Lumsden co-ordinates CAE projects.

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Contact:

CAE
 University of Canterbury
 Private Bag 4800
 Christchurch, New Zealand

Street Address:
 39 Creyke Road
 Christchurch 8004

Telephone: +64-3-364-2478
 Fax: +64-3-364-2069

e-mail: j.blakeley@cae.canterbury.ac.nz
<http://www.cae.canterbury.ac.nz>

Executive Director: John P Blakeley
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Guidelines for the Use of Structural Precast Concrete in Buildings

The First Edition of *Guidelines for the Use of Structural Precast Concrete in Buildings* was published by the Centre for Advanced Engineering (CAE) in 1991 and was reprinted in 1992. The material that led to this widely-used book was produced by a Study Group of the New Zealand Concrete Society and the New Zealand Society for Earthquake Engineering.

Following release of the revised New Zealand Concrete Structures Design Standard NZS 3101 in 1995, it was decided to produce a new Second Edition that would not only reflect the requirements of the new code but would also incorporate appropriate developments of accepted "good practice" within the design and construction industry, and include some of the relevant research that has been produced since the first edition was published.

The principal aspects of the uses of structural precast concrete covered are:

- Precast beams (both shell and solid), precast columns and their jointing;
- Beam-column joints, especially if cast-in-place between precast elements;
- Support and continuity of floor slabs;
- Jointing techniques and connectors, constructability and tolerances;
- Diaphragm actions; and
- Behaviour of certain precast concrete wall systems.

Generally, the emphasis is on buildings rather than civil engineering structures. Although there is a particular focus on the seismic aspects of precast concrete construction, many sections refer to gravity loads as well as volume changes such as creep, shrinkage and thermal actions.

This new edition is intended to assist in providing consistently safe and economical applications of structural precast concrete, and at the same time allow innovation in design and construction to continue.